CHAPTER 7.0 DESIGN DETAILS AND CONSTRUCTION CONDITIONS REQUIRING SPECIAL DESIGN ATTENTION

7.1 INTRODUCTION

This chapter includes:

- Design details for key geotechnical components in the pavement system, including drainage and base layer requirements.
- Compaction of subgrades and unbound pavement layers.
- A general overview of the types of potential subgrade problems.
- Identification and treatment of select, widely occurring special geotechnical challenges, including collapsible or highly compressible soils, expansive or swelling soils, subsurface water and saturated soils, and frost-susceptible soils.
- Detailed guidelines on alternate stabilization methods, which are often used to mitigate special problems.

In this chapter, design details for the specific pavement features of base materials and drainage systems are provided. Compaction, one of the key geotechnical issues in pavement design and construction is also covered.

Special challenges generally relate to poor subgrade conditions that occur due to the type of soil and environmental conditions. In this chapter, the various types of problematic soil conditions are reviewed along with widely occurring specific subgrade problems. Although these problematic conditions can be detected with a detailed subsurface exploration, problematic conditions can potentially go unnoticed if they are located between borings.

7.2 SUBSURFACE WATER AND DRAINAGE REQUIREMENTS

The damaging effects of excess moisture on the pavement have long been recognized. Moisture from a variety of sources can enter a pavement structure. This moisture, in combination with heavy traffic loads and freezing temperatures, can have a profound negative effect on both material properties and the overall performance of a pavement system.

As was shown in Figure 3-3, Chapter 3, moisture in the subgrade and pavement structure can come from many different sources. Water may seep upward from a high groundwater table, or it may flow laterally from the pavement edges and shoulder ditches. Knowledge of groundwater and its movement are critical to the performance of the pavement as well as stability of adjacent sideslopes, especially in cut situations. Groundwater can be especially troublesome for pavements in low-lying areas. Thus, groundwater control, usually through interception and removal before it can enter the pavement section, is an essential part of pavement design.

In some cases, pavements are constructed beneath the permanent or a seasonally high watertable. Obviously, drainage systems must perform or very rapid pavement failure will occur. In such cases, redundancy in the drainage design is used (*e.g.,* installation of underdrains and edgedrains) and, often, some monitoring is used to ensure continual function of the drain system.

Capillary action and moisture-vapor movement are also responsible for water accumulating beneath a pavement structure (Hindermann, 1968). Capillary effects are the result of surface tension and the attraction between water and soil. Moisture vapor movement is associated with fluctuating temperatures and other climatic conditions.

As was previously indicated in Chapter 3, the most significant source of excess water in pavements is typically infiltration through the surface. Joints, cracks, shoulder edges, and various other defects in the surface provide easy access paths for water. A study by the Minnesota Department of Transportation indicates that 40% of rainfall enters the pavement structure (Hagen and Cochran 1995). Demonstration Project 87, *Drainable Pavement Systems*, indicates that surface infiltration is the single largest source of moisture-related problems in PCC pavements (FHWA 1994). Although AC pavements do not contain joints, surface cracks, longitudinal cold joints that crack, and pavement edges provide ample pathways for water to infiltrate the pavement structure.

The problem only worsens with time. As pavements continue to age and deteriorate, cracks become wider and more abundant. Meanwhile, joints and edges become more deteriorated and develop into channels through which moisture is free to flow. The result is more moisture being allowed to enter the pavement structure with increasing pavement age, which leads to accelerated development of moisture-related distresses and pavement deterioration.

7.2.1 Moisture Damage Acceleration

Excessive moisture within a pavement structure can adversely affect pavement performance. A pavement can be stable at a given moisture content, but may become unstable if the materials become saturated. High water pressures can develop in saturated soils when subjected to dynamic loading. Subsurface water can freeze, expand, and exert forces of considerable magnitude on a given pavement. Water in motion can transport soil particles and cause a number of different problems, including clogging of drains, eroding of embankments, and pumping of fines. These circumstances must be recognized and accounted for in the design of a pavement.

The detrimental effects of water on the structural support of the pavement system are outlined by AASHTO (1993), as follows:

- Water in the asphalt surface can lead to moisture damage, modulus reduction, and loss of tensile strength. Saturation can reduce the dry modulus of the asphalt by as much as 30% or more.
- Added moisture in unbound aggregate base and subbase is anticipated to result in a loss of stiffness on the order of 50% or more.
- Modulus reduction of up to 30% can be expected for asphalt-treated base and increase erosion susceptibility of cement or lime treated bases.
- Saturated fine-grain roadbed soil could experience modulus reductions of more than 50%.

As noted in Chapters 3, 4, 5 and 6, modulus is the key pavement design property!

The influence of saturation on the life of the pavement is illustrated in Figure 7-1. The severity factor (shown in the figure) is the anticipated relative damage during wet versus dry periods anticipated for the type of road. As an example, Figure 7-1 shows that if the pavement system is saturated only 10% of its life (*e.g.,* about one month per year), a pavement section with a moderate stability factor will be serviceable only about 50% of its fully drained performance period. Specific distresses caused by excessive moisture within flexible and rigid pavements are summarized in Table 7-1 and 7-2, respectively.

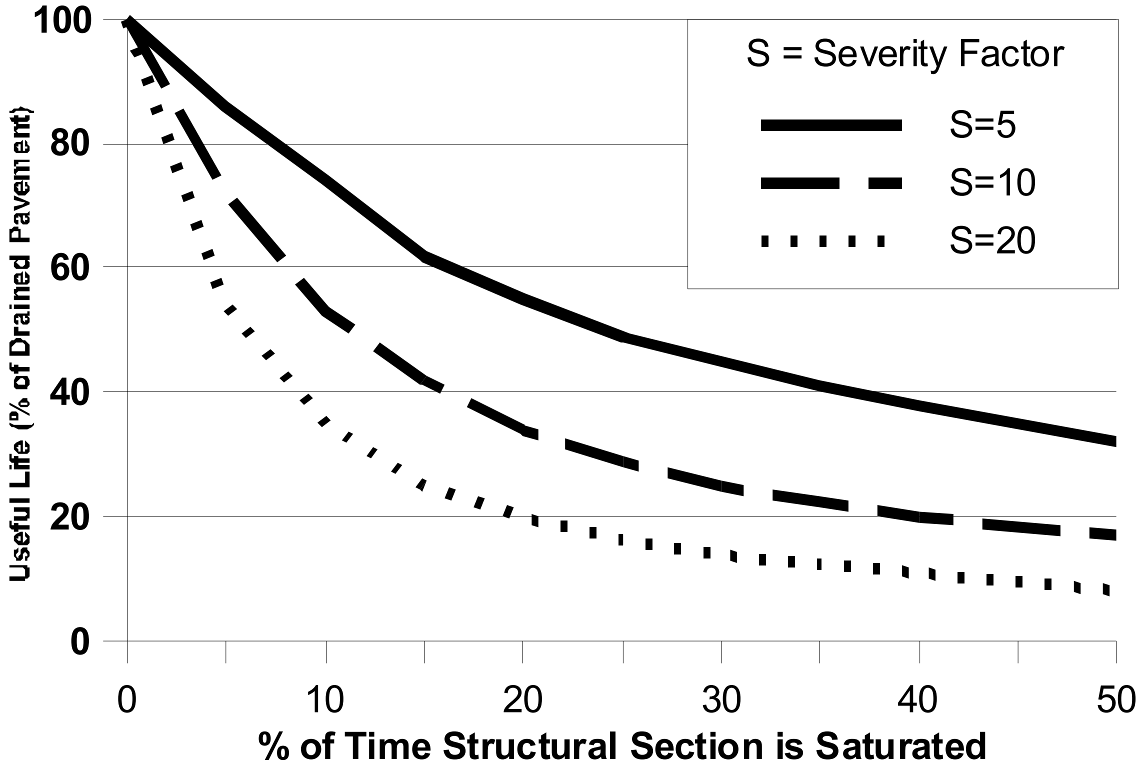

Figure 7-1. The influence of saturation on the design life of a pavement system (after Cedergren, 1987).

Table 7-1. Moisture-related distresses in flexible (AC) pavements (NHI 13126; adapted after Carpenter et al. 1979).

Type	Distress Manifestation	Moisture Problem	Climatic Problem	Material Problem	Load Associated Distress	Structural Defect Begins in		
						AC	Base	Subgrade
Surface Deformation	Bump or Distortion	Excess Moisture	Frost Heave	Volume Increase	No	No	No	Yes
	Corrugation or Rippling	Slight	Moisture and Temperature	Unstable Mix	Yes	Yes	Yes	No
	Stripping	Yes	Moisture	Loss of Bond	No	Yes	No	No
	Rutting	Excess in Granular Layers or Subgrade	Moisture	Plastic Deformation, Stripping	Yes	Yes	Yes	Yes
	Depression	Excess Moisture	Suction & Materials	Settlement, Fill Material	No	No	No	Yes
	Potholes	Excess Moisture	Moisture, Temperature	< Strength > Moisture	Yes	Yes	Yes	Yes
Cracking	Longitudinal	No; Accelerates	No	Construction	No	Faulty Construction	No	No
	Alligator (fatigue)	Yes; Accelerates	Spring-Thaw Strength loss	Thickness	Yes	Yes, Mix	Yes	No
	Transverse	No; Accelerates	Low Temp. Freeze-Thaw Cycles	Thermal Properties	No	Yes, Temp. Susceptible	No	No
	Slippage	Yes	No	Loss of Bond	Yes	Yes, Bond	No	No

Table 7-2. Moisture-related distresses in rigid (PCC) pavements (NHI 13126; adapted after Carpenter et al. 1979).

Type	Distress Manifestation	Moisture Problem	Climatic Problem	Material Problem	Load Associated Distress	Structural Defect Begins in		
						PCC	Base	Subgrade
Surface Defects	Spalling	Possible	Freeze/Thaw Cycles	Mortar	No	Yes	No	No
	Scaling	Yes	Freeze/Thaw Cycles	Chemical Influence	No	Yes; Finishing	No	No
	D-Cracking	Yes	Freeze/Thaw Cycles	Aggregate Expansion	No	Yes	No	No
	Crazing	No	No	Rich Mortar	No	Yes; Weak Surface	No	No
Surface Deformation	Blow-up	No	Temperature	Thermal Properties	No	Yes	No	No
	Pumping and Erosion	Yes	Moisture	Inadequate Strength	Yes	No	Yes	Yes
	Faulting	Yes	Moisture-Suction	Erosion-Settlement	Yes	No	Yes	Yes
	Curling/Warping	Yes	Moisture & Temperature	Moisture and Temperature Differentials	No	Yes	No	No
Cracking	Corner	Yes	Moisture	Cracking Follows Erosion	Yes	No	Yes	Yes
	Diagonal Transverse Longitudinal	Yes	Moisture	Follows Erosion	Yes	No	Yes	Yes
	Punchout (CRCP)	Yes	Moisture	Deformation Follows Cracking	Yes	No	Yes	Yes

7.2.2 Approaches to Address Moisture in Pavements

As was indicated in Chapter 3, to avoid moisture-related problems, a major objective in pavement design should be to keep the base, subbase, subgrade, and other susceptible paving materials from becoming saturated or even exposed to constant high moisture levels. The three approaches described in detail in Chapter 3 for controlling or reducing the problems caused by moisture are

- prevent moisture from entering the pavement system.
- use materials and design features that are insensitive to the effects of moisture.
- quickly remove moisture that enters the pavement system

No single approach can completely negate the effects of moisture on the pavement system under heavy traffic loading over many years. For example, it is practically impossible to completely seal the pavement, especially from moisture that may enter from the sides or beneath the pavement section. While materials can be incorporated into the design that are insensitive to moisture, this approach is often costly and in many cases not feasible (*e.g.,* may require replacing the subgrade). Drainage systems also add cost to the road. Maintenance is required for both drainage systems and sealing systems, for them to effectively perform over the life of the system. Thus, it is often necessary to employ all approaches in combination to obtain the most effective design. The first two approaches involve the surficial pavement materials, which are well covered in the NHI courses on pavement design (*e.g.,* NHI 131060A "Concrete Pavement Design Details and Construction Practices" and the participant's manual) and will not be covered herein. The geotechnical aspects of these approaches include drainage systems for removal of moisture, the requirements of which will be reviewed in the following subsections. Durable base material requirements will be reviewed in the subsequent section, and followed by subgrade stabilization methods to mitigate moisture issues in the subgrade. A method of sealing to reduce moisture intrusion into the subgrade will also be reviewed in the subgrade stabilization section.

7.2.3 Drainage in Pavement Design

Removal of free water in pavements can be accomplished by draining the free water vertically into the subgrade, or laterally though a drainage layer into a system of collector pipes. Generally, the actual process will be a combination of the two (ASSHTO, 1993). Typically in wet climates, if the subgrade permeability is less than 3 m/day (10 ft/day), some form of subsurface drainage or other design features to combat potential moisture problems should be considered. Table 7-3 provides additional climatic conditions and traffic considerations to assist in the assessment of the need for subsurface drainage.

The quality of drainage is defined in both AASHTO 1993 and NCHRP 1-37A based on the principle of time-to-drain. Time-to drain is the time required following any significant rainfall event for a pavement system to drain from a saturated state to a specific saturation or drainage level (*e.g.,* 50% drainage level in AASHTO 1993). The concept can also be applied (at least qualitatively) to other significant moisture events that would saturate the pavement (*i.e.,* flood, snow melt, or capillary rise). The definitions of poor to excellent drainage provided by AASHTO (1993) are given in Table 7-4.

Table 7-3. Assessment of need for subsurface drainage in new or reconstructed pavements (NCHRP 1-37 A, adapted after NHI 13126).

Climatic Condition	Greater than 12 million 20-yr design lane heavy trucks			Between 2.5 and 12 million 20-yr design lane heavy trucks			Less than 2.5 million 20-yr design lane heavy trucks		
	$k_{subgrade}$ (m/day)								
	< 3	3 to 30	> 30	< 3	3 to 30	> 30	< 3	3 to 30	> 30
Wet-Freeze	R	R	F	R	R	F	F	NR	NR
Wet-No Freeze	R	R	F	R	F	F	F	NR	NR
Dry-Freeze	F	F	NR	F	F	NR	NR	NR	NR
Dry-No Freeze	F	NR	NR	NR	NR	NR	NR	NR	NR

LEGEND:

$k_{subgrade}$ = Subgrade permeability.

R = Some form of subdrainage or other design features are recommended to combat potential moisture problems.

F = Providing subdrainage is feasible. The following additional factors need to be considered in the decision making:

(1) Past pavement performance and experience in similar conditions, if any.
(2) Cost differential and anticipated increase in service life through the use of various drainage alternatives.
(3) Anticipated durability and/or erodibility of paving materials.

NR = Subsurface drainage is not required in these situations.

Wet Climate = Annual precipitation > 508 mm (20 in.)

Dry Climate = Annual precipitation < 508 mm (20 in.)

Freeze = Annual freezing index > 83 ºC-days (150 ºF-days)

No-Freeze = Annual freezing index < 83 ºC-days (150 ºF-days)

Table 7-4. AASHTO definitions for pavement drainage recommended for use in both flexible and rigid pavement design (AASHTO, 1993).

Quality of Drainage	Water Removed* Within
Excellent	2 hours
Good	1 day
Fair	1 week
Poor	1 month
Very Poor	Does not Drain

* Based on 50% time-to-drain.

As reviewed in Chapters 3, 5, and 6, drainage effects on pavement performance are incorporated into both the AASHTO 1993 and in the NCHRP 1-37A design methods. In AASHTO 1993, the effect of drainage is considered by modifying the structural layer coefficient (for flexible pavements) and the load transfer coefficient (for rigid pavements) as a function of the quality of drainage and the percent of time the pavement structure is near saturation. The influence of the drainage coefficient (C_d) for rigid pavement design and a drainage modifier (m) for flexible pavement design were demonstrated in the sensitivity studies shown in Chapter 6.

In the NCHRP 1-37A pavement design guide, the impact of moisture on the stiffness properties of unbound granular and subgrade materials is considered directly through the modeling of the interactions between climatic factors (rainfall and temperatures), groundwater fluctuations, and material characteristics of paving layers. Drainage coefficients are not used. However, the benefits of incorporating drainage layers are apparent in terms of distress predictions, which consider seasonal changes in unbound layers and subgrade properties due to moisture and coupled moisture-temperature effects.

Using either the AASHTO 1993 or NCHRP 1-37A method, the influence on design can be significant. For example, in high rainfall areas, the base section of a flexible pavement system (with a relatively thick base layer) can be reduced in thickness by as much as a factor of 2, or the design life extended by an equivalent amount, if excellent drainage is provided versus poor drainage. Likewise, an improvement in drainage leads to a reduction in Portland cement concrete (PCC) slab thickness.

Achieving poor drainage is relatively simple. If the subgrade is not free draining (e.g., not a clean sand or gravel), then the pavement section will require drainage features to drain. Even with edge drainage (*i.e.,* daylighted base or edgedrains), drainage could still be poor. Many designers choose to use dense graded base for its improved construction and presumed structural support over free-draining base. Unfortunately, dense graded base usually does not readily drain and, as a result, structural support will most likely decrease over time.

Due to the low permeability of dense graded base and long drainage path to the edge of the road, drainage in dense graded base is, at best, extremely slow. For example, consider that the permeability of a dense graded base with a very low percentage of fine-grain soil (less than 5% smaller than a 0.075 mm {No. 200 U.S. sieve}) is about 0.3 m/day (1ft/day)(as was reviewed in Chapter 5). Also consider that the length of the drainage path for a two-lane road (lane width of the road draining from the centerline to the edge) is typically 3.7 m (12 ft). An optimistic estimate of the time required to drain a base section that is 300 mm (1 ft) thick and has a slope of 0.02 is 2 days. According to AASHTO definitions of drainage, the pavement section has "good" to "fair" drainage. If the length of the drainage path is two lanes (*i.e.,* 7.3 m {24 ft}), it would take up to a week for the pavement to drain; a condition defined as "fair" drainage (AASHTO, 1993). Base materials often contain more than 5% fines, in which case the permeability and, correspondingly, the drainage can easily be an order of magnitude less than the estimated value for the example (AASHTO, 1993)[1]. In a recent study a Midwestern state found base materials from six different quarries to have 12% to 19% fines and corresponding field permeabilities measured at 2 to 0.01 m/day (7 to 0.03 ft/day) (Blanco et al., 2003). A month or more will then be estimated for pavement drainage; a condition defined as "poor" to "very poor" in AASHTO 1993. In reality, capillary effects and the absence of a driving head of water often cause dense graded base to act like a sponge at low hydraulic gradients. This results in trapped water in the pavement section and "very poor" drainage (*e.g.,* see Dawson and Hill, 1998).

In order to achieve good to excellent drainage, a more permeable, open-graded base and/or subbase will be required, which is tied into a subsurface drainage system. However, this approach only works for new or reconstructed pavements. For existing pavements, retrofitting drainage along the edges of the pavement is the only option, and the existing base material may not drain. However, a significant amount of water can enter the pavement at the crack between the shoulder and the pavement, as well as from lateral movement of water from outside the shoulder. Specific guidelines do not exist currently for retrofit pavements, as only limited data are available. Local experience should be used in selecting pavement candidates for retrofitting. Performance of similar retrofitted sections, if available, can be a valuable tool in the decision making process.

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[1] Based on hydraulic conductivity tests, AASHTO notes a decrease in permeability from 3 m/day (10 ft/day) with 0% fines down to 0.02 m/day (0.07 ft/day), with the addition of only 5% non-plastic fines and (0.0003 m/day (0.001 ft/day) with 10% non-plastic fines. An additional order of magnitude decrease was observed with base containing plastic fines.

7.2.4 Types of Subsurface Drainage

In the past, pavement systems were designed without any subdrainage system. These sections are commonly labeled "bathtub" or "trench" sections because infiltrated water is trapped in the base and subbase layers of the pavement system.

Many types of subsurface drainage have been developed over the years to remove moisture from the pavement system. These subsurface drainage systems can be classified into several groups. One criterion for classifying various subsurface drainage systems is the source of moisture that the system is designed to control. For example, a **groundwater control system** refers to a subsurface drainage system designed to remove and control the flow of groundwater. Similarly, an **infiltration control system** is designed to remove water that seeps into the pavement structural section. A **capillary break system** is designed to intercept and remove rising capillary water and vapor movement.

Probably the most common way to classify a subsurface drainage system is in terms of its location and geometry. Using this classification, subsurface drainage systems are typically divided into five distinct types:

- Longitudinal edgedrains.
- Transverse and horizontal drains.
- Permeable bases.
- Deep drains or underdrains.
- Interceptor drains.

Each type may be designed to control several sources of moisture and may perform several different functions. In addition, the different types of subsurface drainage system may be used in combination to address the specific needs of the pavement being designed. Drains constructed primarily to control groundwater general consist of underdrains and/or interceptor drains. The interceptor drains are usually placed outside the pavement system to intercept the lateral flow of water (*e.g.,* from cut slopes) and remove it before it enters the pavement section. Deep underdrains (greater than 1 m {3 ft} deep) are usually installed to lower the groundwater level in the vicinity of the pavement. The design and placement of these interceptor and underdrains should be addressed as part of the geotechnical investigation of the site.

Edgedrains placed in trenches under the shoulders at shallower depths are used to handle water infiltrating the edge of the pavement from above. Edgedrains are combined with permeable base and, in some cases, transverse and horizontal drains to form a drainable pavement system to control surface infiltration water. Drainable pavement systems generally consist of the following design features (as shown in Figure 7-2):

- a full-width permeable base under the AC- or PCC-surfaced travel lanes,
- a separator layer under the permeable base to prevent contamination from subgrade materials,
- longitudinal edgedrains with closely spaced outlets. An alternative to closely spaced outlets is dual drainage systems with parallel collector drains. An alternative to edgedrains is daylighting directly into a side ditch.

Designs not incorporating these combinations of features cannot be expected to function properly. Drainage systems for new construction and rehabilitation are described in more detail in the following sections.

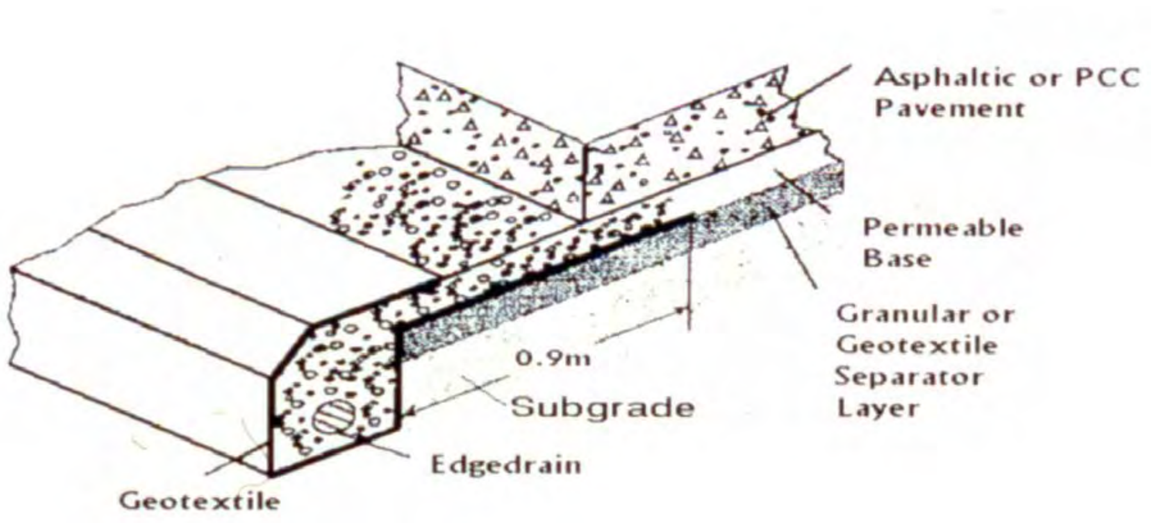


Figure 7-2. Design elements of a drainable pavement system (after FHWA, 1992).

7.2.5 Daylighted Base Sections

Daylighted bases were one of the first attempts to remove surface infiltration water from the pavement system. The original daylighted base consists of a dense-graded aggregate base that extends to the ditchline or side slope. Daylighted dense-graded bases are expected to intercept water that infiltrates through the pavement surface and drain the water through the base to the ditch. However, most dense-graded daylighted bases are slow draining and, therefore, not very effective in removing infiltrated water.

This situation led to the development of a new generation of daylighted bases—daylighted permeable bases (Fehsenfeld 1988), as illustrated in Figure 7-3. Several studies have reported that daylighted permeable bases are as effective in removing infiltrated water and reducing moisture-related distresses as permeable bases with edgedrains (Yu et al. 1998b). However, they require regular maintenance because the exposed edge of daylighted bases easily becomes clogged with fines, soil, vegetation, and other debris. Also, stormwater from ditch lines can easily backflow into the pavement structure. Further study into daylighted permeable bases is needed to verify long-term performance of this design.

7.2.6 Longitudinal Edgedrains

Longitudinal edgedrains consist of a drainage system that runs parallel to the traffic lane. The edgedrains collect water that infiltrates the pavement surface and drains water away from the pavement through outlets. Four basic types of edgedrains systems have been used:

- pipe edgedrains in an aggregate filled trench,
- pipe edgedrains with porous concrete (*i.e.,* cement treated permeable base) filled trench,
- prefabricated geocomposite edgedrains in a sand backfilled trench, and
- aggregate trench drain ("French" drain).

The most commonly used edgedrain is a perforated pipe varying in diameter from 100 – 150 mm (4 – 6 in.). The pipe is generally situated in an aggregate trench to allow water to flow toward the pipe. Another type of edgedrain that is often used in rehabilitation projects is a geocomposite drain in a sand filled trench with pipe outlets. Typical cross sections of edgedrains are illustrated in Figures 7-5 and 7-6.

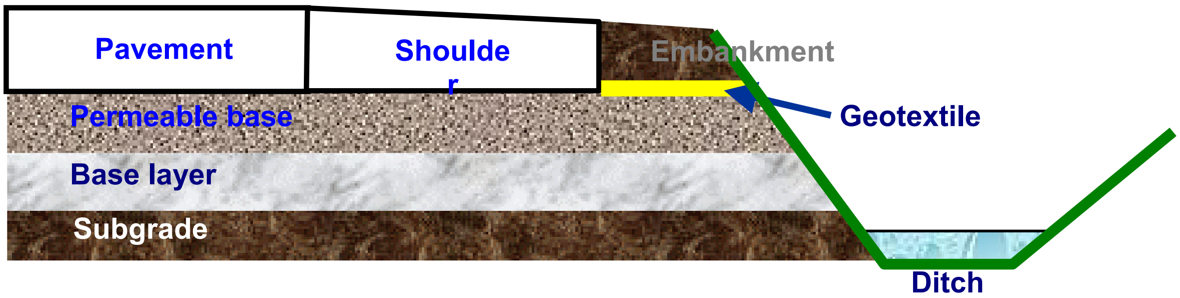


Figure 7-3. Typical AC pavement with a daylighted base.

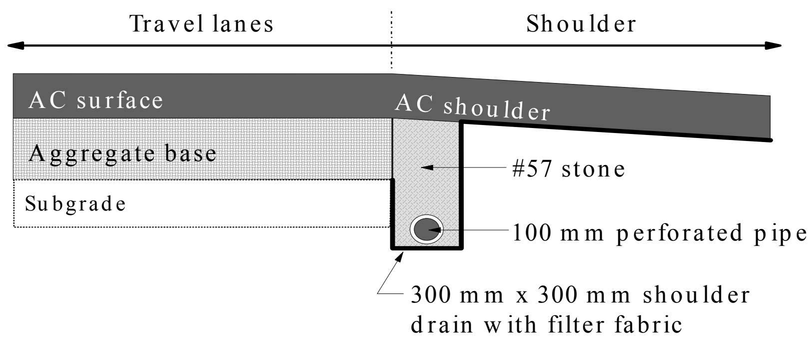


Figure 7-4. Typical AC pavement with pipe edgedrains (ERES, 1999).

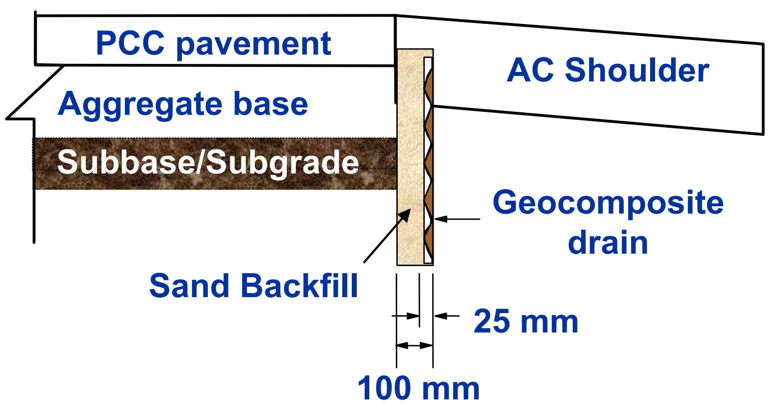


Figure 7-5. Typical PCC pavement with geocomposite edgedrains (ERES, 1999).

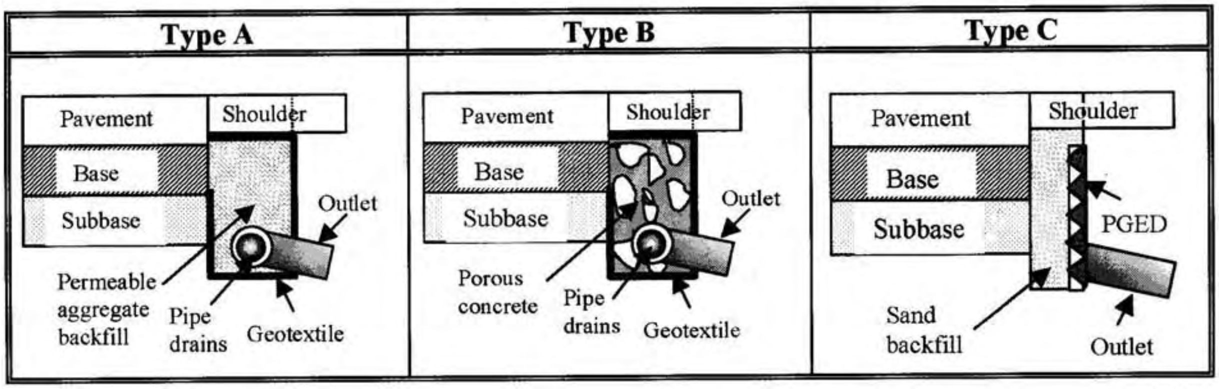


Figure 7-6. Typical edgedrains for rehabilitation projects (NCHRP 1-37A).

The effectiveness of longitudinal edgedrains depends on how they are used. Longitudinal edgedrains can be effective if used with other drainage features. Typical application of edgedrains include the following:

- New construction
  - Longitudinal edgedrains (pipe or geocomposite) with nonerodible dense-graded bases*.
  - Longitudinal edgedrains (pipe or geocomposite) with permeable bases.
- Existing pavement
  - Retrofit longitudinal edgedrains (pipe or geocomposite) with nonerodible dense-graded bases*.
- Rehabilitation projects
  - Retrofit longitudinal edgedrains (pipe or geocomposite) with nonerodible dense-graded bases*. (On projects using rubblized base or dense graded base with erodible fines, the geotextile filter in the trench should not be placed between the base and the edgedrain aggregate to avoid clogging the geotextile filter - see Figure 7-6.)

* *Retrofit edgedrains are usually not recommended for pavements with dense-graded aggregate bases containing more than 15% fines (fraction passing the 0.075 mm (No. 200 sieve)). Excessive fines can clog the drains, and the loss of fines through the pipes can lead to significant base erosion. As previously indicated, dense graded base with greater than 5% fines is not anticipated to freely drain, however edgedrains can be used to effectively remove water entering the longitudinal joint (or crack) between the pavement and the shoulder.*

The field performance of edgedrains installed without a permeable base has been mixed. Some studies show little or no benefit, but others report significant improvement in pavement performance. Considering that only about 30% of all edgedrains in-service are functioning properly, mostly due to improper construction (Daleiden 1998; Sawyer 1995), the mixed report is not surprising. In many cases, the outlet pipes are crushed during construction or clogged due to inadequate maintenance. The performance of edgedrains placed in untreated dense-graded base sections seems to be dependent to a significant degree on local climatic conditions, natural drainage characteristics, subgrade type, pavement design, construction and construction inspection, and maintenance. Longitudinal edgedrains with permeable bases have been found to be effective in draining pavements and reducing moisture-related distresses when well designed, constructed, and maintained.

The type of geocomposite edgedrains used also affects performance. Older versions did not have sufficient hydraulic capacity and had not been recommended for draining permeable bases. However, some of the geocomposites available today do provide sufficient hydraulic capacity to drain permeable bases. The main disadvantage of geocomposite edgedrains is that they are difficult to maintain.

The use of aggregate trench drains, however, is not recommended because of low hydraulic capacity and inability to be maintained. An exception might be permeable cement stabilized aggregate placed in a trench.

The size of the longitudinal perforated pipe in the edgedrain is often based on maintenance requirements for cleaning capabilities and reasonable distance between outlets. Maintenance personnel should be consulted before finalizing these dimensions. The smallest diameter suitable for cleaning is 75 mm (3 in.), however many state highway agencies and the FHWA suggest a minimum pipe size of 100 mm (4 in.) based on maintenance considerations (FHWA, 1992). FHWA also recommends a maximum outlet spacing of 75 m (250 ft).

One of the most critical items for edgedrains is the grade of the invert. Construction control of very flat grades is usually not possible, leaving ponding areas that result in subgrade weakening and premature failures. Although not a popular concept, it may be more economical to raise the pavement grade to develop adequate drain slopes for the subsurface drainage facilities (*e.g.,* Florida). To achieve a desirable drainage capacity, a minimum slope may be required for the edgedrain that is greater than the slope of the road. However, this requirement may not be practical, and the pipe will mostly be sloped the same as the roadway. It is suggested that rigorous maintenance be anticipated, especially when adequate slopes cannot be achieved (FHWA, 1992).

The ditch or storm drain pipe must be low and large enough to accept the inflow from the edgedrain without backup. FHWA recommends the outlet be at least 150 mm (6 in.) above the ten-year storm flow line of the ditch or structure. The outlet should also be at a location and elevation that will allow access for maintenance activities (both cleaning and repair). Outlets and shallow pipes should be located well away from areas of expected future surface maintenance activities, such as sign replacement and catch basin cleanout or repair. FHWA also recommends angled or radius outlet connections to facilitate clean out and video inspection. Outlet headwalls, typically precast concrete, are also an essential part of the edgedrain system to prevent displacement of the outlet pipe and crushing during roadway and ditchline maintenance operations. Locations of guardrail, sign, signal, and light posts must be adjusted to prevent damage to the subsurface drainage facilities.

An offset dual pipe with a large diameter parallel collector drain line is an alternative to decrease the number of outlets (see Figure 7-7). The large diameter collector pipe (either heavy walled plastic or concrete) runs either adjacent to or below a perforated drainage pipe, as shown in Figure 7-7, to facilitate quick removal of subsurface water. The collector pipe can outflow into culverts or stormwater systems. Manholes can be installed for cleanout. These systems are quite common in Europe and have been used by a few U.S. agencies to reduce outlet maintenance issues (*e.g.,* California and, experimentally, in Kentucky).

7.2.7 Permeable Bases

A permeable base is designed to rapidly move surface infiltration water from the pavement structure to the side ditch through longitudinal edgedrains with outlets or by daylighting directly into the side ditch. Permeable bases contain no fines (0% passing the 0.075-mm (No. 200) sieve) to allow easy flow of water. In order to meet excellent drainage requirements (*i.e.,* time-to-drain of less than 2 hours from Table 7-4), permeable bases typically are required to have permeability values in excess of 300 m/day (1000 ft/day) and thicknesses of 100 mm (4 in.) (as recommended by FHWA, 1992). The performance of permeable base layers meeting these requirements will be demonstrated later in Section 7.2.12 on design of pavement drainage.

The structural capacity of angular, crushed aggregate permeable base, with a percentage of two-face crushing, is usually equivalent to the structural capacity of an equal thickness of dense-graded base. However, in order to meet these hydraulic requirements, a coarse uniform gravel must be used, which is often difficult to construct. Asphalt or cement treatments are often used to stabilize the gravel for construction, as discussed in Section 7.3. While stabilizing the base with a cement or asphalt binder will initially offer greater structural support than dense-graded base, it should be remembered that the primary purpose of the stabilizer is to provide stability of the permeable base during the construction phase. It is generally assumed that the binder will either break down or be removed by stripping with time. Thus, increase in structural support is generally not assumed for stabilized aggregate.

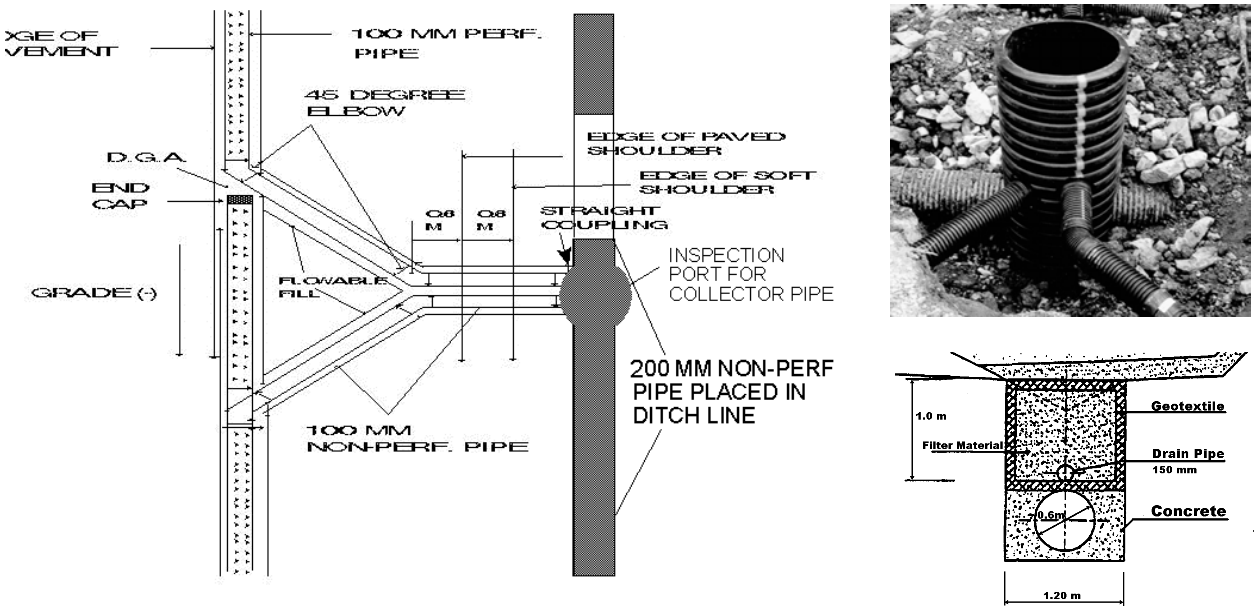


Figure 7-7. Dual pipe edgedrain systems showing alternate locations of the parallel collector pipe, either adjacent to or beneath the drain line (Christopher, 2000).

Typical cross-sections of AC and PCC pavements with permeable bases were illustrated in Figures 7-4, 7-5, and 7-6. Note that a geotextile filter should be wrapped around a portion of the trench, but not over the interface between the permeable base and drainage aggregate.

7.2.8 Dense-Graded Stabilized Base with Permeable Shoulders

This system consists of a nonerodible dense-graded base, typically lean concrete base (LCB) or asphalt treated base (ATB), under the traffic lanes and a permeable base under the shoulder. Longitudinal edgedrains are placed in the permeable base course to carry the excess moisture from the pavement structure. The recommended design for a dense-graded stabilized base with permeable shoulders is illustrated in Figure 7-8. This design offers better support under the traffic lanes where it is needed most, while still providing a means to remove water from the pavement structure. This design is now required for all high-type PCC pavements (pavements designed for more than 2.5 million equivalent single axle loads {ESALs}) in California (CALTRANS 1995).

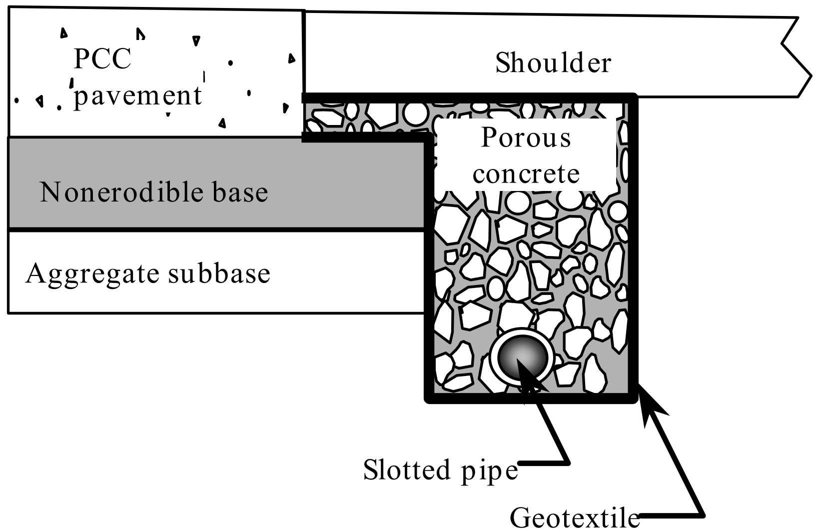


Figure 7-8. Recommended design of PCC pavement with a nonerodible dense-graded base and permeable shoulders (ERES, 1999).

7.2.9 Horizontal Geocomposite Drains

Several states (*i.e.,* Maine, Wisconsin, and Virginia) have experimented with the use of horizontal geocomposite drains, with properties sufficient to handle the estimated flow and support traffic loads, placed either below or above dense graded base, placed as a drainage layer beneath full depth asphalt, or placed between a crack and seat concrete surface and a new asphalt layer. When placed below the base aggregate, the geocomposite shortens the drainage path and reduces the time-to-drain. When placed directly beneath the pavement surface, the geocomposite intercepts and removes infiltration water before it enters the base and/or subgrade. The geocomposite is tied into an edgedrain system. Systems using this technology have been found to have excellent drainage (*i.e.,* time-to-drain of less than 2 hours from Table 7-4). Additional information, including preliminary performance information, is reported by Christopher et al. (2002).

7.2.10 Separator Layers

Separator layers play an essential role in the performance of a pavement with a permeable base by preventing fines in the underlying layers and subgrade soils from infiltrating into the permeable base, thus maintaining the permeability and effective thickness of the base course. Various combinations of materials have been used as separator layers, including the following (FHWA 1994a):

- Dense-graded aggregate (most used by far)
- Geotextiles
- Cement-treated granular material
- Asphalt chip seals
- Dense-graded asphalt concrete

These materials have been used with varying degrees of success. Lime- or cement-treated subgrades alone are not acceptable as separator layers over fine-grained soils. There have been some classic failures of lime-treated soils used as separator layers in which pumping into the permeable base caused excessive settlements.

According to a survey of 33 states, 27 used dense-graded aggregates or asphalt-treated mixtures as separator layers on a regular basis. Sixteen states used geotextiles sparingly, and 11 states used either dense-graded material or geotextiles as separator layers (Yu et al., 1998). Generally, a dense-graded aggregate or a dense-graded AC material separator layer is preferred over a geotextile for competent subgrades because the aggregate layer will provide a strong construction platform and distribute traffic loads to the subgrade. However, geotextile separator layers have been used directly beneath base layers where the additional support of a subbase is not required. For sensitive subgrades that are easily disturbed by construction (*e.g.,* silts and saturated cohesive soils), a geotextile separator layer used in conjunction with a granular subbase minimizes disturbance and provides a good construction platform. Geotextile separators also allow the use of a more open-graded, freer-draining subbase, reducing the potential for subbase saturation. Geotextiles can also be used as a separator layer in conjunction with compacted or treated subgrades, or granular subbases. If appropriately design, geosynthetics can also be used to increase subgrade support, as reviewed later in Section 7.6.5.

7.2.11 Performance of Subsurface Drainage

Many studies have shown the benefits of subsurface drainage in terms of improved performance. Cedergren (1988) believes that all important pavements should have internal drainage, claiming drainage eliminates damage, increases the life of the pavement, and is cost-effective.

Moisture-related damage to pavements has become more significant as traffic loadings have increased over the past 40 years. The annual rate of ESAL applications has virtually doubled every 10 years, causing tremendous problems related to moisture accelerated damage. A pavement may be adequately drained for one level of traffic, but as traffic increases, moisture-related damage may increase greatly. As a result, more and more states have begun to employ subsurface drainage systems (Yu et al. 1998b). Many preliminary studies indicate that drainage systems are indeed beneficial in terms of reducing certain types of pavement deterioration. However, due to some instances of poor design, construction, and/or maintenance, all have not performed as well as expected.

One example of unsatisfactory performance is some early cracking observed on a few PCC pavements with permeable bases. This occurs for a variety of reasons, including:

- Inadequate design of permeable bases and separator layers.
- Inadequate edgedrains.
- Lack of quality control during construction, such as inadequate joint sawing. Sometimes the concrete from the slab enters the permeable base, creating a thicker slab than was originally designed. Joints must be sawed deeper to ensure the proper depth is obtained to cause cracking through the joint.
- Lack of maintenance of the drainage system after the highway is open to traffic.
- Possible settlement of the PCC slab over untreated aggregate permeable bases.

Permeable bases must be constructed of durable, crushed aggregate to provide good stability through aggregate interlock. They must have a separator layer capable of preventing the pumping of fines into the permeable base from underlying layers and from preventing any intermixing of the permeable base and separator layer. Permeable bases must also have pipe edgedrains to drain the infiltrated water with suitable outlets at reasonable outlet spacing or must be daylighted directly into the ditch. Finally, to ensure good performance, the drainage system must be regularly maintained.

7.2.12 Design of Pavement Drainage

Design of pavement drainage consists of determining:

1. The hydraulic requirements for the permeable layer to achieve the required time-to-drain.
2. The edgedrain pipe size and outlet spacing requirements.
3. Either the gradation of requirements for a graded aggregate separation layer or the opening size, permeability, endurance, and strength requirements for geotextile separators.
4. The opening size, permeability, endurance, and strength requirements for geotextile filters, or the gradation of the granular filters (to be used in the edgedrain).

The following provides an outline of the design steps and procedures required for the design of each of these subsurface drainage components. Complete design details and supporting information can be found in NHI 13126 on Pavement Subsurface Drainage Design – Reference Manual (ERES, 1999).

7.2.13 Hydraulic Requirements for the Permeable Layer(s)

Basically there are two approaches to the hydraulic design of a permeable layer:

1. Time-to-drain
2. Steady-state flow.

The time-to-drain approach was previously discussed in Section 7.2.3 and simply means the time required for a percentage of the free water (*e.g.,* 50%) to drain, following a moisture event where the pavement section becomes saturated. In the steady-state flow approach, uniform flow conditions are assumed, and the permeable layer is designed to drain the water that infiltrates the pavement surface. The time-to-drain approach will be the basis for design in this manual, as it is currently the procedure recommended by the FHWA, AASHTO, and NCHRP 1-37A for pavement design. Elements of steady state flow will be used to determine outlet spacing. (For additional discussion of steady state flow methods see FHWA, 1992 and ERES, 1999.)

The time-to-drain approach assumes the flow of water into the pavement section until it becomes saturated (the drainage layer plus the material above the drainage layer). Excess precipitation will not enter the pavement section after it is saturated; this water will simply run off the pavement surface. After the rainfall event, the drainage layer will drain to the edgedrain system. Engineers must design the permeable layer to drain relatively quickly to prevent the pavement from being damaged.

A time-to-drain of 50% of the drainable water in 1 hour is recommended as a criterion for the highest class roads with the greatest amount of traffic (FHWA, 1992). For most other high use roadways, a time-to-drain of 50% of the drainable water in 2 hours is recommended. For secondary roads, a minimum target value of 1 day is recommended (U.S. Army Corps of Engineers, 1992). Remember, in all cases, the goal of drainage is to remove all drainable water as quickly as possible.

The time-to-drain is determined by the following equation:

$$t = T \times m \times 24 \qquad \textit{Eq. 7.1}$$

where,
- t = time-to-drain in hours
- T = Time Factor
- m = "m" factor (see Eq. 7.3)

A simplified design chart for determining a time-to-drain of 50% time factor, T_{50}, is provided in Figure 7-9. This chart was developed for one degree (*i.e.,* direction) of drainage and is adequate for most designs. For expanded charts to cover additional degrees of drainage and desired percent drained see FHWA, 1992 and ERES, 1999.

The time factor is based on the geometry of the drainage layer (e.g., the permeable base layer). The geometry includes the resultant slope (S_R) and length (L_R); the thickness of the drainage layer (H), which is the length the water must travel within a given layer; and, the percent drained (U), (i.e., 50%). S_R and L_R are based on the true length of drainage and are determined by finding the resultant of the cross and longitudinal pavement slopes (S_X and S, respectively) and lengths (L_x and L, respectively). The resultant length is measured from the highest point in the pavement cross-section to the point where drainage occurs (*i.e.,* edgedrain or daylighted section). First, the slope factor (S_l) must be calculated:

$$S_1 = \frac{L_R\, S_R}{H} \qquad \textit{Eq. 7.2}$$

where, S_R	=	$(S^2 + S_X^2)^{1/2}$
L_R	=	$W\,[1 + (S/S_X)^2]^{1/2}$
W	=	width of permeable layer in m (ft)
H	=	thickness of permeable layer in m (ft)
1 ft	=	0.3 m

Figure 7-9 is then entered with the S_l, and the resulting T_{50} to be used in Eq. 7.1 is determined.

The "m" factor in Eq 7.1 is determined by the equation:

$$m = \frac{N_o\, L_R^{\ 2}}{kH} = \frac{N_o\, L_R^{\ 2}}{\psi} \qquad \textit{Eq.}7.3$$

where, N_o	=	the effective porosity of the drainage layer
k	=	permeability of drainage layer in m/day (ft/day)
H	=	thickness of drainage layer in m (ft)
ψ	=	the transmissivitty of the drainage layer in m^2/day (ft^2/day)
1 ft	=	0.3 m

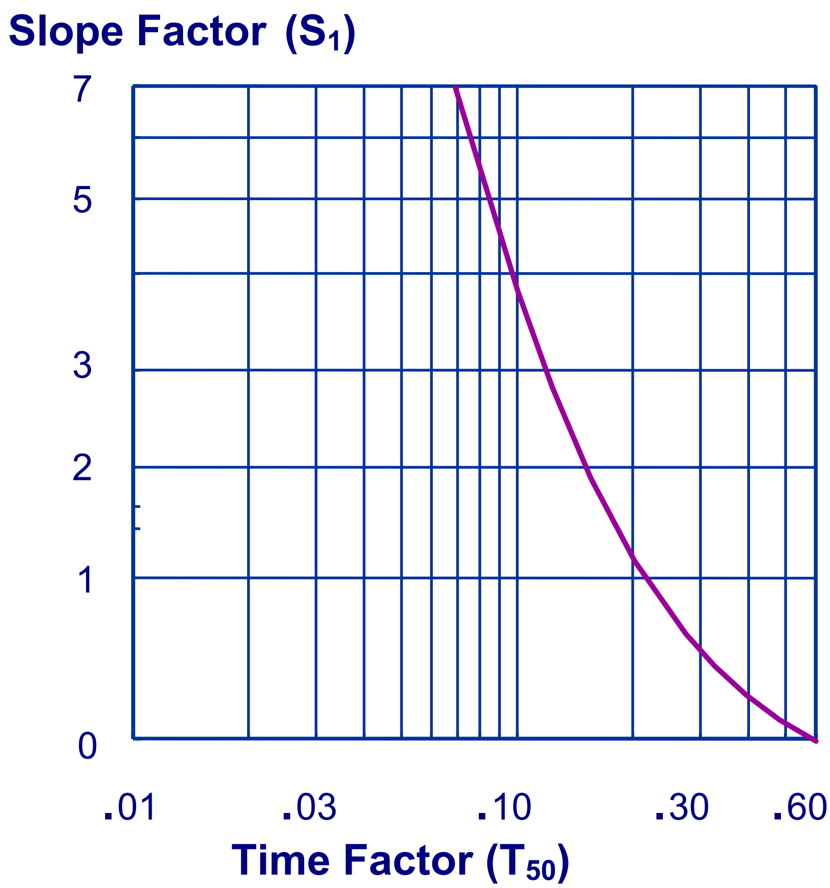


Figure 7-9. Time Factor for 50% Drainage (ERES, 1999).

The intrinsic factors that represent the drainage capabilities of drainage layer base are represented by the effective porosity (N_o) and the coefficient of permeability (k) or, if H is known, the transmissivity of the drainage layer. The effective porosity is the ratio of the volume of water that can drain under gravity from the material to the total volume of the material. It is a measure of the amount of water that can be drained from a material. The value can be easily determined by saturating a sample of material and measuring the amount of water that drains. Additional information on the determination of these characteristics for aggregate drainage layers are covered in detail in FHWA, 1992 and NHI 13126.

For example, using the recommended 4-inch-thick open-graded base layer with a permeability of 300 m/day (1000 ft/day) at a cross slope of 2% in a relatively flat (1% grade) road alignment would produce the following time-to-drain for a four lane road draining from the center (W = 7.3 m (24 ft)):

$$S_R = (S^2 + S_X^2)^{½} = (0.01^2 + 0.02^2)^{½} = 0.022$$

$$L_R = W((1 + (S/S_X)^2)^{½} = 24 \text{ ft } [1 + (0.01/0.02)^2]^{½} = 26.8 \text{ ft}$$

$$S_l = (L_R\ S_R)/H = (26.8 \text{ ft} \times 0.022)/\ 0.33 \text{ ft} = 1.8$$

$$m = (N_e L_R^2) / (kH) = (0.25 \times (26.8 \text{ ft})^2)/(1000 \text{ ft/day} \times 0.33 \text{ ft}) = 0.54 \text{ days}$$

From Figure 7-9 with $S_l = 1.8$, $T = 0.16$

Therefore, $t = T \times m \times 24 = 0.16 \times 0.54$ days $\times$ 24 hrs/day = 2.1 hrs

Since the time-to-drain is close to 2 hrs, the drainage layer would provide excellent drainage, as defined in Table 7-4.

According to a sensitivity analysis on time-to-drain performed in ERES, 1999, time-to-drain is most sensitive to changes in the coefficient of permeability and the resultant slope, decreasing exponentially with increasing permeability and slope values. Time to drain increases linearly with increasing length and effective porosity, while thickness has very little effect.

The DRIP microcomputer program developed by FHWA can be used to rapidly evaluate the effectiveness of the drainage system and calculate the design requirements for the permeable base design, separator, and edgedrain design, including filtration requirements. The program can also be used to determine the drainage path length based on pavement cross and longitudinal slopes, lane widths, edgedrain trench widths (if applicable), and cross-section geometry crowned or superelevated. The software can be downloaded directly from the FHWA WEB page http://www.fhwa.dot.gov/pavement/library.htm and is included with the NCHRP 1-37A pavement design software.

7.2.14 Edgedrain Pipe Size and Outlet Spacing Requirements

The FHWA recommends a minimum pipe diameter of 100 mm (4 in.) and a maximum outlet spacing of 75 m (250 ft) to facilitate cleaning and video inspection. The adequacy of these requirements can be confirmed by evaluating the anticipated infiltration rate or, more conservatively, from the maximum flow capacity of the drainage layer.

With the flow capacity method, the estimated discharge rate from drainage layer is determined. For example, the conventional 100-mm (4-in.) thick open-graded base layer with a permeability of 300 m/day (1000 ft/day) used in the previous time-to-drain example provides excellent drainage for most conditions (FHWA, 1992). This 100-mm (4-in.) thick free-draining base layer has a transmissivity (*i.e.,* permeability multiplied by the thickness) of about 28 m^2/day (300 ft^2/day). For a typical roadway gradient of 0.02 (for a 2% grade), the open-graded base layer has a flow capacity of 0.13 ft^3/day (6 ft^3/day) per ft length of road. Thus at an outlet spacing of 75 m (250 ft), the quantity of flow at the discharge (Q) of the edgedrain system would be 33 m^3/day (1500 ft^3/day).

The capacity of a circular pipe flowing full can be determined by Manning's equation:

$$Q = \frac{53.01}{n} D^{8/3} S^{1/2} \qquad Eq. 7.4$$

where,	Q	= Pipe capacity, cu ft/day
	D	= Pipe diameter, in.
	S	= Slope, ft/ft
	n	= Manning's roughness coefficient
		= 0.012 for smooth pipe
		= 0.024 for corrugated pipe
	1 ft	= 0.3 m
	1 in	= 25.4 mm

Thus, for a 100-mm (4-in.) smooth wall pipe at a 1% grade, the flow capacity is 504 m^3/day (17800 ft^3/day), which is more than adequate to handle the maximum quantity of flow anticipated for the edgedrain system. However, the 100-mm (4-in.) pipe is still recommended to facilitate inspection and cleaning.

In the infiltration method, a design rainfall and an infiltration ratio are selected. Pavement infiltration is determined by the equation

$$q_i = C \times R \times 1/12 \text{ (ft/in)} \times 24 \text{ (hr/day)} \times 1 \text{ ft} \times 1 \text{ ft)} \qquad Eq. 7.5$$

which can be simplified to:

$$q_i = 2\,C\,R \qquad Eq. 7.5a$$

where,	q_i	= Pavement infiltration, ft^3/day/ft^2 of pavement
	C	= Infiltration ratio
	R	= Rainfall rate, in./hr

The infiltration ratio C represents the portion of rainfall that enters the pavement through joints and cracks. The following design guidance for selecting the infiltration coefficient is suggested (FHWA, 1992):

Asphalt concrete pavements	0.33 to 0.50
Portland cement concrete pavements	0.50 to 0.67

To simplify the analysis and provide an adequate design, FHWA suggest using a value of 0.5. The design storm whose frequency and duration will provide an adequate design must be selected. A design storm of 2-year frequency, 1-hour duration, is suggested. Figure 7-10 provides a map of generalized rainfall intensity.
The analysis is then performed by substituting into the above equation for the specific region of the country. The drainage layer discharge rate q_d can then be determined by multiplying the infiltration rate by the resultant length of the pavement section L_R as follows:

$$q_d = q_i L_R \qquad \textit{Eq. 7.6}$$

This discharge rate can then be compared to the flow capacity of the drainage layer and the lower value of the two used to evaluate the outlet spacing and pipe size.

7.2.15 Separator Layer

As indicated in the previous section, the separator consists of a layer of aggregate material (treated or untreated) or a geotextile layer placed between the permeable base and the subgrade or other underlying layers. The separator layer has to maintain separation between permeable base and subgrade, and prevent them from intermixing and support construction traffic. It may also be desirable to use a low permeable layer that will deflect water from the permeable base horizontally toward the pavement edge (NCHRP 1-37A).

If dense-graded aggregate separator layers are used, the aggregate must be a hard, durable material. Based on FHWA guide specifications for materials selection and construction of aggregate separation layers, the aggregate should meet the following requirements:

- The aggregate should have at least two fractured faces, as determined by the material retained on the 4.74 mm (No. 4) sieve; preferably, it should consist of 98% crushed stone.
- The L.A. abrasion wear should not exceed 50%, as determined by AASHTO T 96, *Resistance to Abrasion of Small Size Coarse Aggregate by Use of Los Angeles Machine*.

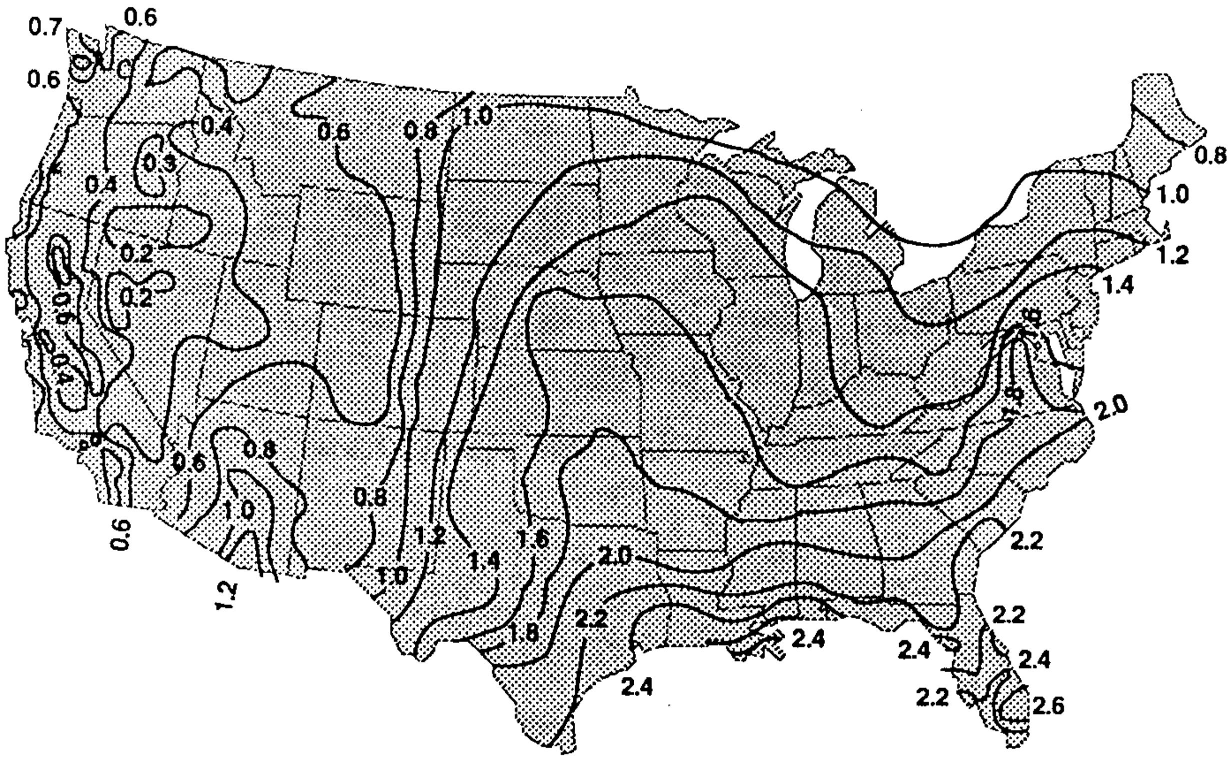


Figure 7-10. Rainfall Intensity in in./hr for a 2-year, 1-hour Storm Event (FHWA, 1992).

- The soundness loss percent should not exceed 12 or 18%, as determined by the sodium sulfate or magnesium sulfate tests, respectively. The test shall be in accordance with AASHTO T 104, *Soundness of Aggregate by the Use of Sodium Sulfate or Magnesium Sulfate*.
- The gradation of this layer should be such that it allows a maximum permeability of approximately 5 m/day (15 ft/day) with less than 12% of the material passing the 0.075 mm (No. 200) sieve, by weight.
- Material passing the 425 mm (No. 40) sieve shall be nonplastic, in accordance with AASHTO T 90, “Determining the Plastic Limit and Plasticity Index of Soils.”

7.2.16 Geotextile Separator and Filter Design

As a separator, just as with the granular layer, the geotextile must prevent the intermixing of the permeable base and the adjacent subgrade or subbase layer. Also as with aggregate separator layers, the geotextile layers will have to satisfy filtration criteria. In order to provide support for construction traffic, the geotextile must also satisfy survivability and endurance criteria. Additional requirements for subgrade improvement are reviewed in Section 7.6.5. Both woven and non-woven geotextiles have been used for the separation application. The criteria for filtration and survivability are outlined in the following paragraphs and are basically the same as that required for the edgedrain geotextile filters. The only notable exception is that the separation layer can have a much lower permeability (compatible with the subgrade) than the edgedrain filter (compatible with the permeable base).

As a filter for the edgedrain, the geotextile must be designed to allow unimpeded flow of water into edgedrain system over the life of the system. The geotextile must prevent soil from washing into the system without clogging over time. The FHWA presents three basic principles for geotextile design and selection (Holtz et al., 1998):

1. If the larger pores in the geotextile filter are smaller than the largest particles of soil, these particles will not pass the filter. As with graded granular filters, the larger particles of soil form a filter bridge on the geotextile, which, in turn, filters the smaller particles of the soil. Thus, the soil is retained and particle movement and piping is prevented.
2. If the smaller openings in the geotextile are sufficiently large so that the smaller particles of soil are able to pass through the filter, then the geotextile will not clog.
3. A large number of openings should be present in the geotextile so that proper flow can be maintained even if some of the openings later become clogged.

The geotextile filtration characteristics must be checked for compatibility with the gradation and permeability of the subgrade. The requirements for proper performance can be appropriately selected by using the following design steps.

Step 1. Determine the gradation of the material to be separated/filtered. The filtered material is directly above and below the geocomposite drainage layer. Determine D_{85}, D_{15} and percent finer than a 0.075 mm (No. 200) sieve.

Step 2. Determine the permeability of the base or subbase $k_{base/subbase}$, whichever is located directly above the geocomposite drainage layer. (For placement directly beneath the hot-mix or PCC pavement applications, the default permittivity requirement will be used.

Step 3. Apply design criteria to determine apparent open size (AOS), permeability (k), and permittivity (ψ) requirements for the geotextile (after Holtz et al., 1998)

$AOS \leq D_{85\ base/subbase}$ (For woven geotextile)

$AOS \leq 1.8\ D_{85\ subgrade}$ (For nonwoven geotextile)*

$k_{geotextile} \geq k_{base/subbase}$
$\psi \geq 0.1\ sec^{-1}$

* *For noncohesive silts and other highly pumping susceptible soils, a filter bridge may not develop, especially considering the potential for dynamic, pulsating flow. A conservative (smaller) AOS $\leq D_{85\ subgrade}$ is advised, and laboratory filtration tests are recommended.*

Step 4. In order to perform effectively, the geotextile must also survive the installation process. AASHTO M288 (1997) provides the criteria for geotextile strength required to survive construction of roads, as shown in Table 7-5. Use Class 2 where a moderate level of survivability is required (*i.e.,* for subgrade CBR > 3, where at least 150 mm (6 in.)) of base/subbase and normal weight construction equipment is anticipated, and where filters are used in edgedrains). Class 1 geotextiles are recommended for CBR < 3 and when heavy construction equipment is anticipated. For separation layers, a minimum of 150 mm (6 in.) of base/subbase should be maintained between the wheel and geotextile at all times.

In projects using recycled concrete, rubblizing, or crack-and-seat techniques, geotextiles and granular filters are susceptible to clogging by precipitate and should not be indiscriminately used to separate the permeable base from the drain or wrapped around pipes. Geotextiles should not be placed between the recycled material and the drain, but could be placed beneath and on the outside of the drain to prevent infiltration of the subgrade and subbase layers (see Figure 7-2.)

Table 7-5. Geotextile survivability requirements (AASHTO M 288-96).

Test	Test Method	Units	Geotextile Class			
			Class 1		Class 2	
			< 50%*	≥ 50%*	< 50%*	≥ 50%*
Grab Strength	ASTM D 4632	N	1400	900	1100	700
Seam Strength	ASTM D 4632	N	1200	810	990	630
Tear Strength	ASTM D 4533	N	500	350	400	250
Puncture Strength	ASTM D 4833	N	500	350	400	250
Burst Strength	ASTM D 3786	kPa	3500	1700	2700	1300

*Note: Elongation measured in accordance with ASTM D 4632 with < 50% typical of woven geotextiles and ≥ 50% typical of nonwoven geotextiles. (1 N = 0.22 lbs, 1 kPa = 0.145 psi)

7.3 BASE LAYERS: REQUIREMENTS, STABLILIZATION & REINFORCEMENT

The function of the base course varies according to the type of pavement, as was described in Chapter 1. Under rigid pavements, the base course is used to: (1) provide uniform and stable support, (2) minimize damaging effects of frost action, (3) provide drainage, (4) prevent pumping of fine-grained soils at joints, (5) prevent volume change of the subgrade, (5) increase structural capacity of the pavement, and (6) expedite construction. Under flexible pavements, the prime function of the base course is to structurally improve the load-supporting capacity of the pavement by providing added stiffness and resistance to fatigue, as well as to provide a relatively thick layer to distribute the load through a finite thickness of pavement. The base may also provide drainage and give added protection against frost action where necessary.

To meet these functional requirements, the base course as a minimum should have the following characteristics:

- To prevent pumping, a base course must be either free draining or it must be highly resistant to the erosive action of water. Erodibility is covered in more detail in the next section.
- To provide drainage, the base course may or may not be a well-graded material, but it should contain little or no materials finer than a 0.075 mm (No. 200) sieve. It may sometimes be stabilized with asphalt or cement.
- A base course design for frost action should be non-frost susceptible and free draining.
- To improve resistance to deformation and improve structural support or reduce the thickness, it may be desirable to stabilize the base course with asphalt or cement, as reviewed in Section 7.3.2 and 7.3.3, or to reinforce it with geosynthetics, as reviewed in Section 7.3.4.
- A base course need not be free draining to provide structural capacity, but it should be well-graded and should resist deformation due to loading.

The aggregate used for base must be hard, durable material. As a minimum, the aggregate should meet the following requirements:

- The aggregate should have at least two fractured faces; preferably, it should consist of 98% crushed stone.
- The L.A. abrasion wear should not exceed 45% as determined by AASHTO T 96, *Resistance to Abrasion of Small Size Coarse Aggregate by Use of Los Angeles Machine*.

- The soundness loss percent should not exceed 12 or 18%, as determined by the sodium sulfate or magnesium sulfate tests, respectively. The test should be performed in accordance with AASHTO T 104, *Soundness of Aggregate by the Use of Sodium Sulfate or Magnesium Sulfate* (see Chapter 5).
- For permeable base, the gradation of this layer should enable free movement of water with a minimum permeability value around 300 m/day (1,000 ft/day) (see Section 7.2) and the material passing the 0.425 mm (No. 40) sieve should be non-plastic in accordance with AASHTO T 90, *Determining the Plastic Limit and Plasticity Index of Soils*.

7.3.1 Erodibility of Bases

Preventing significant erosion of the base and subbase materials is very important for the control of moisture-related distresses, such as pumping and faulting in JPCP and punchouts in CRCP, as discussed in NCHRP 1-37A. Erodibility is the loss of base material due to hydraulic action, most often at the joints in rigid pavements, but also along the edge of both rigid and flexible pavements. The condition is related to the durability of the base in relation to its potential to break down under dynamic traffic loads, climatic conditions, environmental effects, as well as water action. As truck traffic increases, a more erosion resistant base is required, along with more adequate joint load transfer design (*e.g.,* use of dowels in joints). Traffic level is a very critical factor in the consideration of base/subbase course erosion, especially considering that the base/subbase under PCC slabs of reconstructed projects will likely receive 10 to 20 times more load repetitions over their design life than in the past.

While the base course is the layer most often affected by erosion, any layer directly beneath a treated base can experience serious erosion. There are many examples of the erosion of fine grained soils beneath a stabilized base course causing loss of support and joint faulting. Thus, some agencies now place a dense graded granular subbase layer between the base and compacted subgrade to reduce this problem. Other agencies stabilize the top layer of a fine-grained soil with lime to reduce this problem; however, this approach must produce a sufficiently hard material with adequate compressive strength and uniformity along the project. Geotextiles are also used as separation layers to hold the subgrade materials in place. Another alternative that has been used successfully is to place a layer of recycled crushed PCC beneath the dense treated base.

The NCHRP 1-37A guide provides guidance for assessing the erodibility potential of various materials used in new JPCP and CRCP design and in PCC overlays of existing flexible or rigid pavements. The effect of erosion is considered empirically in the form of erodibility classification assessment for specific design levels. The design procedure provides the framework for which erosion can be considered on a more mechanistic basis in the future (such as iterative month-by-month damage accumulation, and inclusion of Level 1 laboratory erosion test). Tables 7-6, 7-7, and 7-8 provide the Material Classification requirements for Level 1, Level 2, and Level 3 design, respectively.

7.3.2 Bound Bases

In order to achieve the highest erodibility levels, stabilized base or subbase materials often produced by the addition of a sufficient quantity of stabilizing agent (usually cement or asphalt) to produce materials with significant tensile strength (*e.g.,* Erodibility Class 1a in Table 7-7). Such materials are considered to be bound bases and have a substantial increase in structural capacity over that of unbound and modified (treated) bases. Bound bases or subbases are not considered to be geotechnical materials, and are not covered in this manual. Users are referred to NHI courses on pavements (*e.g.,* NHI 131033) for additional information.

Table 7-6. Level 1 recommendation for assessing erosion potential of base material (NCHRP 1-37A).

Erodibility Class	Material Description and Testing
Class based on the material type and test results	Test not fully developed for nationwide uses; thus Level 1 cannot be implemented at this time. The tests currently being considered to assess the erodibility of paving materials include - Rotational shear device for cohesive or stabilized materials (Bhatti et al., 1996). - Jetting test (Bhatti et al., 1996). - Linear and rotational brush tests (Dempsey, 1982). - South African erosion test (DeBeer, 1990).

Table 7-7. Design Level 2 recommendations for assessing erosion potential of base material (*NCHRP 1-37A adapted after the Permanent International Association of Road Congresses, PIARC, 1987*).

Erodibility Class	Material Description and Testing
1	(a) Lean concrete with approximately 8% cement; or with long-term compressive strength > 17.2 MPa (2,500 psi) [> 13.8 MPa (2,000 psi) at 28-days] and a granular subbase layer or a stabilized soil layer or a geotextile fabric is placed between the bound base and subgrade; otherwise Class 2. (b) Hot mixed asphalt concrete with 6% asphalt cement that passes appropriate stripping tests and aggregate tests and a granular subbase layer or a stabilized soil layer; otherwise Class 2. (c) (c) Permeable drainage layer (asphalt-treated aggregate or cement-treated aggregate) and with an appropriate granular or geotextile separation layer placed between the treated permeable base and subgrade.
2	(a) Cement-treated granular material with 5% cement manufactured in-plant, or long-term compressive strength 13.8 to 17.2 MPa (2,000 to 2,500 psi) [10.3 MPa to 13.8 MPa (1,500 to 2,000 psi) at 28-days] and a granular subbase layer or a stabilized soil layer or a geotextile fabric is placed between the treated base & subgrade; otherwise Class 3. (b) (b) Asphalt-treated granular material with 4% asphalt cement that passes appropriate stripping test and a granular subbase layer or a treated soil layer or a geotextile is placed between the treated base and subgrade; otherwise Class 3.
3	(a) Cement-treated granular material with 3.5% cement manufactured in-plant, or with long-term compressive strength 6.9 MPa to 13.8 MPa (1,000 to 2,000 psi) [5.2 MPa to 10.3 MPa (750 to 1,500 psi) at 28-days]. (b) Asphalt-treated granular material with 3% asphalt cement that passes appropriate stripping test.
4	Unbound crushed granular material having dense gradation and high quality aggregates.
5	Untreated soils (PCC slab placed on prepared/compacted subgrade).

Table 7-8. Design Level 3 recommendations for assessing erosion potential of base material based on material description only (NCHRP 1-37A).

Erodibility Class	Material Description and Testing
1	(a) Lean concrete with previous outstanding past performance and a granular subbase layer or a stabilized soil layer or a geotextile layer is placed between the treated base and subgrade; otherwise Class B. (b) Hot mixed asphalt concrete with previous outstanding past performance and a granular subbase layer or a stabilized soil layer is placed between the treated base and subgrade; otherwise Class B. (c) Permeable drainage layer (asphalt- or cement-treated aggregate) and a granular or a geotextile separation layer between the treated permeable base and subgrade. Unbonded PCC Overlays: HMAC separation layer (either dense or permeable graded) is specified.
2	(a) Cement-treated granular material with good past performance and a granular subbase layer or a stabilized soil or a geotextile layer is placed between the treated base and subgrade; otherwise Class C. (b) Asphalt-treated granular material with good past performance and a granular subbase layer or a stabilized soil layer or a geotextile is placed between the treated base and subgrade; otherwise Class C.
3	(a) Cement-treated granular material that has exhibited some erosion and pumping in the past. (b) Asphalt-treated granular material that has exhibited some erosion and pumping in the past. Unbonded PCC Overlays: Surface treatment or sand asphalt is used.
4	Unbound crushed granular material having dense gradation and high quality aggregates.
5	Untreated subgrade soils (compacted).

7.3.3 Modified (or Treated) Bases

The addition of cement or asphalt (typically less than 5%) to stabilize unbound base or subbase with the primary purpose of improving the stability for construction are considered to be modified or treated bases. Modified materials are usually considered to behave structurally as unbound granular material. These bases or subbases are considered to be geotechnical materials. Stabilization is most often required for open graded (permeable) bases (OGB), which tend to rut and weave under construction activities. Tables 7-9 and 7-10 provide the recommendations for asphalt-treated bases and cement-treated bases, respectively.

The strength of cement-treated bases will depend in part on adequate curing during construction. The mixture must be well compacted at optimum moisture content, and adequate density must be obtained throughout the layer. Density control will also be important for the uniformity of asphalt-treated base materials. Although stabilization is often used to reduce the thickness of the base, it should be recognized that thin bases (less than 150 mm (6 in.) thickness) are often extremely difficult to construct to the exact depth, creating the potential for very thin base layers in localized areas. Construction of thin bases requires a very competent subgrade or a good working platform (as reviewed in Section 7.6). Construction quality control for cement- and asphalt-treated materials is reviewed in Chapter 8.

Table 7-9. Recommended asphalt stabilizer properties for asphalt-treated permeable base/subbase materials.

Specification	Requirement	Test Method
Aggregate	(a) hard, durable material with at least two fractured faces; preferably, consisting of 98% crushed stone. (b) L.A. abrasion wear should not exceed 45%. (c) Soundness loss percent should not exceed 12 as determined by the sodium sulfate, or 18% by the magnesium sulfate tests.	Visual Classification AASHTO T 96 AASHTO T 104, *Soundness of Aggregate by the Use of Sodium Sulfate or Magnesium Sulfate*
AC content	AC content must ensure that aggregates are well coated. Minimum recommended AC content is between 2.5 – 3% by weight. Final AC content should be determined according to mix gradation and film thickness around the coarse aggregates.	ASTM D 2489, *Test Method for Degree of Particle Coating of Bituminous-Aggregate Mixtures.*
AC grade	A stiff asphalt grade (typically 1 grade stiffer than the surface course is recommended).	Penetration, viscosity, or Superpave binder testing can be performed to determine AC grade.
Anti-stripping	Anti-stripping test should be performed on all AC treated materials.	AASHTO T283, *Resistance of Compacted Bituminous Mixture to Moisture Induced Damage.*
Anti-stripping Agents	Aggregates exhibiting hydrophilic characteristics can be counteracted with 0.5 – 1% lime.	NCHRP Report 274.
Permeability	Minimum mix permeability: 300 m/day (1000 ft/day).	AASHTO T 3637, *Permeability of Bituminous Mixtures.*

Table 7-10. Recommended Portland cement stabilizer properties for cement-treated permeable base/subbase materials.

Specification	Requirement	Test Method
Aggregate	(a) Hard, durable material with at least two fractured faces; preferably, consisting of 98 percent crushed stone. (b) (a) L.A. abrasion wear should not exceed 45%. (c) (c) Soundness loss percent should not exceed 12 or 18%, as determined by the sodium sulfate or magnesium sulfate tests, respectively.	Visual Classification AASHTO T 96-94 AASHTO T 104-86, "Soundness of Aggregate ...Use of Sodium Sulfate or Magnesium Sulfate"
Cement	Portland cement content selected must ensure that aggregates are well coated. An application rate of 130 to 166 kg/m^3 (220 to 285 lb/yd^3) is recommended.	Must conform to the specification of AASHTO M 85, *Portland Cement*
Water-to-cement ratio	Recommended water-to-cement ratio to ensure strength and workability: 0.3 to 0.5.	
Workability	Mix slump should range between 25 – 75 mm (1 – 3 in.).	
Cleanness	Use only clean aggregates	
Permeability	Minimum mix permeability: (300 m/day) 1,000 ft/day.	

7.3.4 Base Reinforcement

A more recent form of stabilization is the use of geosynthetics (primarily geogrids) to reinforce the base for flexible pavement systems, which has been found under certain conditions to provide significant improvement in performance of pavement sections. The principal effect of reinforcement in base-reinforced flexible pavements is to provide lateral confinement of the aggregate layer. Lateral confinement arises from the development of interface shear stresses between the aggregate and the reinforcement, which, in turn, transfers load to the reinforcement. The interface shear stress present when a traffic load is removed continues to grow with traffic load applications, meaning that the lateral confinement of the aggregate increases with increasing load applications. Increases in traffic volume up to a factor of 10 to reach the same distress level (25-mm (1 in.) rutting) have been observed for reinforced sections, versus unreinforced sections of the same design asphalt and base thickness (Berg et al., 2000). Table 7-11 provides a summary of the conditions for which various geosynthetic products should be considered for this application.

Table 7-11. Qualitative review of reinforcement application potential for paved permanent roads (after Berg et al., 2000).

Roadway Design Conditions		Geosynthetic Type					
Subgrade	Base / Subbase Thickness[1] (mm)	Geotextile		Geogrid [2]		GG-GT Composite	
		Nonwoven	Woven	Extruded	Knitted or Woven	Open-Graded Base[3]	Well-Graded Base
Soft (CBR < 3) (M_R <30 MPa)	150 – 300	❹	●	●	❑	●	⑤
	> 300	❹	❹	◗	◗	◗	⑤
Firm - Vy. Stiff (3 ≤ CBR ≤ 8) (30 ≤ MR ≤ 80)	150 – 300	❍	◗	●	❑	●	⑤
	> 300	❍	❍	❍	❍	❍	❍

KEY: ● — usually applicable ◗ — applicable for some conditions
❍ — usually not applicable ❑ — insufficient information at this time ⑤ — see note

NOTES: 1. Total base or subbase thickness with geosynthetic reinforcement. Reinforcement may be placed at bottom of base or subbase, or within base for thicker (usually > 300 mm (12 in.)) thicknesses. Thicknesses less than 150 mm (6 in.) not recommended for construction over soft subgrade. Placement of less than 150 mm (6 in.) over a geosynthetic not recommended.
2. For open-graded base or thin bases over wet, fine grained subgrades, a separation geotextile should be considered with geogrid reinforcement.
3. Potential assumes base placed directly on subgrade. A subbase also may provide filtration.
❹ Reinforcement usually applicable, but typically addressed as a subgrade stabilization.
⑤ Geotextile component of composite likely is not required for filtration with a well-graded base course; therefore, composite reinforcement usually not applicable.

Current design methods for flexible pavements reinforced with a geosynthetic in the unbound aggregate base layer are largely empirical methods based on a limited set of design conditions over which test sections have been constructed (*i.e.,* AASHTO 4E-SR Standard of Practice Guidelines for Base Reinforcement). These design methods have been limited in use due to 1) absence of nationally recognized reinforced base design procedure, 2) narrow range of test section design conditions from which the method was calibrated, and 3) proprietary design methods pertaining to a single geosynthetic product. Recently FHWA sponsored a study to develop an interface for including geosynthetic base reinforcement in mechanistic empirical design, consistent with the NCHRP 1-37A model. This work is currently in review, but shows excellent promise for the incorporation of these methods into pavement design.

In the interim, AASHTO 4E includes a design approach that relies upon the assessment of reinforcement benefit as defined by a Traffic Benefit Ratio (TBR) or a Base Course reduction Ratio (BCR). TBR is defined as the ratio of the number of traffic loads between an otherwise identical reinforced and unreinforced pavement that can be applied to reach a particular permanent surface deformation of the pavement. BCR defines the percentage reduction in the base course thickness of a reinforced pavement such that equivalent life (*e.g.,* surface deformation) is obtained between the reinforced and the unreinforced pavement with the greater aggregate thickness. The philosophy of this approach is one in which applicability of the technology and reinforcement benefits are assessed by empirical considerations. Reinforcement benefit defined in this manner is then used to modify an existing unreinforced pavement design.

The proposed design procedure in AASHTO 4E follows the steps listed below:

Step 1. Initial assessment of applicability of the technology.
Step 2. Design of the unreinforced pavement.
Step 3. Definition of the qualitative benefits of reinforcement for the project.
Step 4. Definition of the quantitative benefits of reinforcement (TBR or BCR).
Step 5. Design of the reinforced pavement using the benefits defined in Step 4.
Step 6. Analysis of life-cycle costs.
Step 7. Development of a project specification.
Step 8. Development of construction drawings and bid documents.
Step 9. Construction of the roadway.

Step 1 involves assessing the project-related variables given in Table 7-11 and making a judgment on whether the project conditions are favorable or unfavorable for reinforcement to be effective and what types of reinforcement products (as defined in Table 7-11) are appropriate for the project.

Step 2 involves the design of a conventional unreinforced typical pavement design cross section or a series of cross sections, if appropriate, for the project. Any acceptable design procedure can be used for this step.

Step 3 involves an assessment of the qualitative benefits that will be derived by the addition of the reinforcement. The two main benefits that should be assessed are whether the geosynthetic will be used for an extension of the life of the pavement (*i.e.,* the application of additional vehicle passes), a reduction of the base aggregate thickness, or a combination of the two. Berg et al. (2000) has listed additional secondary benefits that should also be considered.

Step 4 is the most difficult step in the design process and requires the greatest amount of judgment. This step requires the definition of the value, or values, of benefit (TBR and/or BCR) that will be used in the design of the reinforced pavement. The definition of these benefit values for a range of design conditions is perhaps the most actively debated and most currently studied topic within this field. Given the lack of a suitable analytical solution for the definition of these terms, Berg et al. (2000) has suggested that these values be determined by a careful comparison of project design conditions, as defined in previous steps, to conditions present in studies reported in the literature. The majority of these studies have been summarized in Berg et al. (2000) in a form that allows direct comparison to known project conditions. In the absence of suitable comparison studies, an experimental demonstration method involving the construction of reinforced and unreinforced pavement test sections has been suggested and described in Berg et al. (2000), and may be used for the definition of benefit for the project conditions. The reasonableness of benefit values should be carefully evaluated such that the reliability of the pavement is not undermined.

Step 5 involves the direct application of TBR or BCR to modify the unreinforced pavement design defined in Step 2. TBR can be directly used to define an increased number of vehicle passes that can be applied to the pavement, while BCR can be used to define a reduced base aggregate thickness such that equal life results. Within the context of an AASHTO pavement design approach, it is possible to calculate a BCR knowing a TBR and vise versa for the specific project design conditions, however this approach has not been experimentally or analytically validated.

With the unreinforced and reinforced pavement designs defined, a life-cycle cost analysis should be performed to assess the economic benefit of reinforcement. This step will dictate whether it is economically beneficial to use the geosynthetic reinforcement. Remaining steps involve the development of project specifications, construction drawings, bid documents, and plans for construction monitoring. Berg et al. (2000) has presented a draft specification that may be adopted for this application.

Even though the application of geosynthetic reinforcement of flexible pavements has been proposed and examined over the past 20 years, research in this area is quite active, meaning that new design methods should be expected in the near future. These new design methods will hopefully provide less empirical methods for assessing reinforcement benefit and be expressed as a function of the variables that are known to influence benefit.

7.4 COMPACTION

Compaction of the subgrade, unbound base, and subbase materials is a basic design detail and is one of the most fundamental geotechnical operations for any pavement project. Compaction is used to increase the stiffness and strength, decrease the permeability, and increase the erosion resistance of geomaterials. Compaction can also reduce the swelling potential for expansive soils. Thus, the intent of compaction is to maximize the soil strength (and minimize the potential volume change) by the proper adjustment of moisture and the densification at or near the ideal moisture content, as discussed in this section.

In most instances, once heavy earthwork and fine grading is completed, the uppermost zone of subgrade soil (roadbed) is improved. The typical improvement technique is by means of water content adjustments and densification by compaction. Higher density requirements are routinely established for the top two feet of at-grade roadbeds and for embankments. The soil in cut areas may need to be undercut and backfilled to obtain the strength and uniformity desired. Heavy proof rolling equipment (270 to 450 kN (30 to 50 tons)) can be used to identify areas of non-uniform support in prepared subgrades. Proofrolling and other field construction aspects of compaction are covered in Chapter 8. Perhaps the most common problem arising from deficient construction is related to moisture-density control, which can be avoided or at least minimized with a thorough plan and execution of the plan as it relates to QC/QA during construction, as reviewed in Chapter 8. This plan should pay particular attention to proper moisture content, proper lift thickness for compaction, and sufficient configuration (e.g., weight and width) of the compaction equipment utilized.

7.4.1 Compaction Theory

The basic engineering principles of soil compaction date back to work by Proctor in the 1930s. Compaction can be performed in the laboratory using static, kneading, gyratory, vibratory, or impact compactors. Each method has its advantages and disadvantages, but impact compaction using a falling hammer is the standard in practice today. Standard laboratory compaction tests are described in more detail in Chapter 5. In these tests, soil is mixed with water at a range of moisture contents w and compacted using a specified compaction energy (e.g., ft-lbs/ft^3 or joules/m^3). Figure 7-11 illustrates the effect of compaction energy on laboratory compaction curves. As described in Chapter 5, the Modified Proctor compaction test (ASTM D1557/ AASHTO T-180) has a compaction energy of 2,700 kN-m/m^3 (56,000 ft-lb/ft^3), which is nearly 5 times the compaction energy of 600 kN-m/m^3 (12,400 ft-lb/ft^3) in the Standard Proctor test (ASTM D698/AASHTO T-99). Likewise, increased compaction energy in the field will increase the maximum dry unit weight and decrease the associated optimum water content.

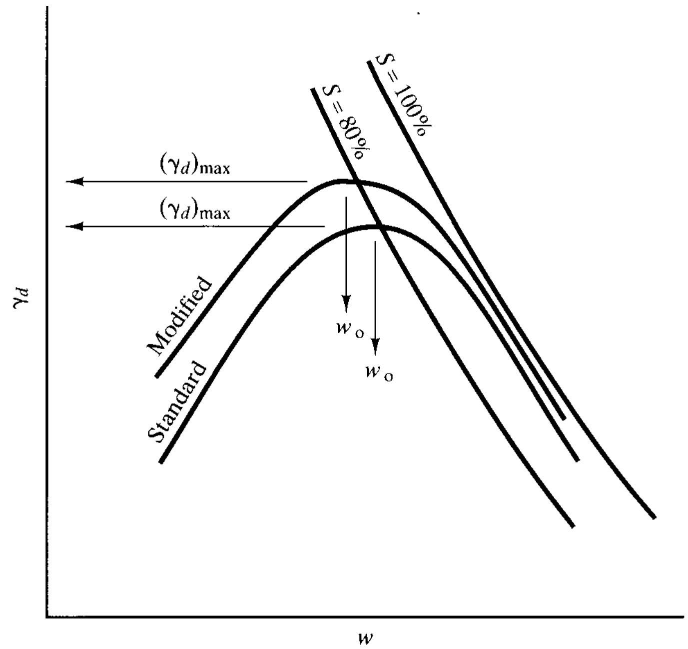


Figure 7-11. Effect of compaction energy on compaction curves (Coduto, 1999).

Different soils will generally have differently shaped compaction curves. This fact will aid in identifying the corresponding laboratory curve for materials encountered in the field. Figure 7-12 shows typical compaction curves for several different soils. Coarser, granular soils typically have fairly steep compaction curves, with large changes in density for small changes in moisture content, while highly plastic clays exhibit fairly flat compaction curves. The maximum dry density is higher for coarser soils and the optimum moisture content is lower. Some cohesionless soils will also exhibit two peaks in the compaction curve; one at very dry conditions, where there are no capillary tensions to resist the compaction effort, and the other at the optimum moisture content, where optimum lubrication between particles occurs.

Nearly all compaction specifications are based on achieving a minimum dry unit weight in the field. This is usually expressed in terms of the relative compaction C_R:

$$C_R = \frac{\gamma_d}{(\gamma_d)_{\max}} \times 100\% \qquad \text{Eq. 7-5}$$

in which γ_d is the dry unit weight achieved in the field and $(\gamma_d)_{max}$ is the maximum dry unit weight as determined from a specified laboratory compaction test.

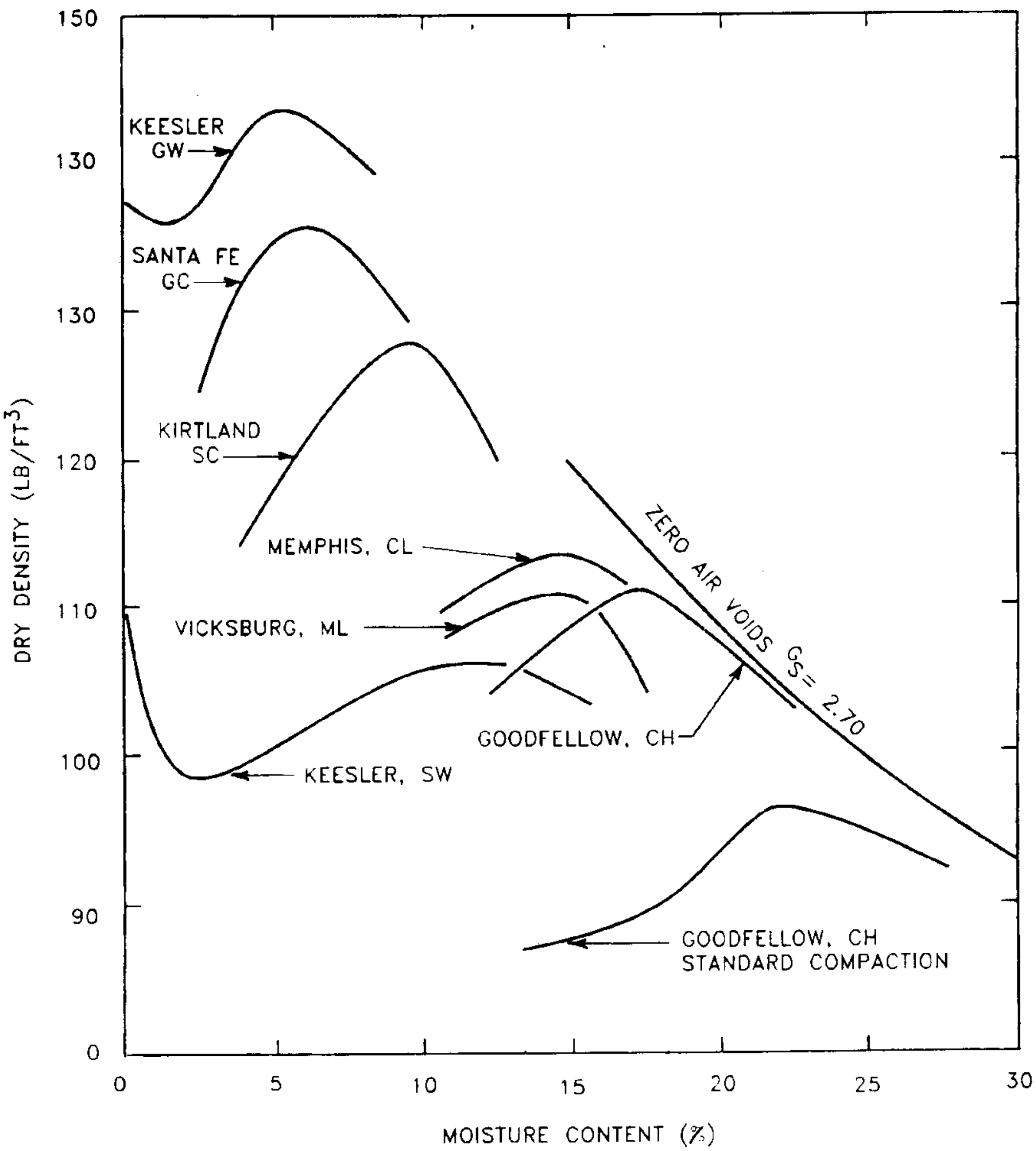


Figure 7-12. Laboratory compaction curves for different soils (Rollings and Rollings, 1996).

The water content at compaction is also sometimes specified because of its effect on soil fabric, especially for clays. Clays compacted dry of optimum have a flocculated fabric (see Figure 7-13), which generally corresponds to higher permeability, greater strength and stiffness, and increased brittleness. Conversely, clays compacted wet of optimum to the same equivalent dry density tend to have a more oriented or dispersed fabric, which typically corresponds to lower permeability, lower strength and stiffness, but more ductility.

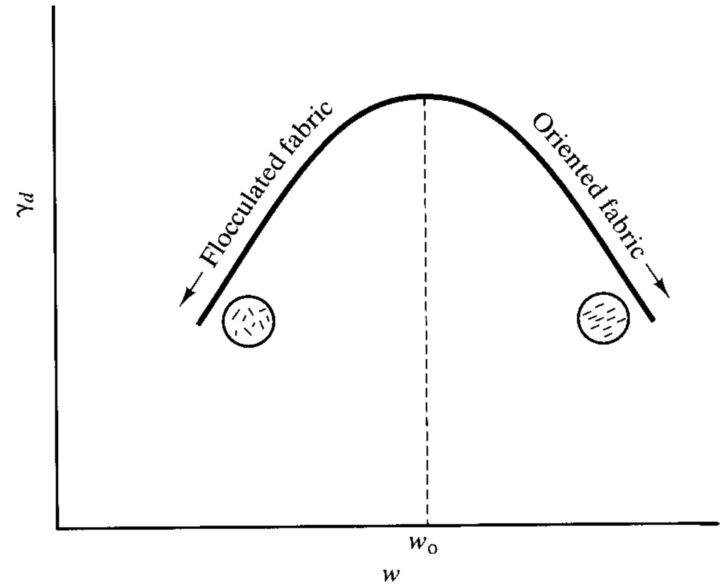


Figure 7-13. Effect of compacted water content on soil fabric for clays (Coduto, 1999).

7.4.2 Effect on Soil Properties

The principal effects of compaction on soil properties are as follows:

- *Density:* As described in the preceding sections, the most direct measurable effect of compaction is an increase in soil density. Typical laboratory values of maximum dry density values and optimum moisture contents for different soils were summarized in Chapter 5, Table 5-18 and 5-19.
- *Strength:* Intuitively, one expects strength to increase with compaction energy and to be larger at low water contents than at high values. Figure 7-14 summarizes typical strength versus water content and compaction energy for a lean clay where strength is quantified by CBR (Rollings and Rollings, 1996). The data in the figure generally confirm intuitive expectations. The strength dry of the optimum water content is larger for higher compaction energies, as expected, and is up to an order of magnitude higher than the strength when compacted wet of optimum. Note, however, that higher compaction energies can produce slightly lower strength values when a fine-grained soil is compacted at water contents higher than the optimum. **Also note that the strength in the figure is based on unsaturated soils. If material compacted dry of optimum becomes saturated, a significant decrease in strength can occur, with strengths even less than that of the same soil compacted wet of optimum**. Large changes in strength upon wetting are associated with fine-grained silts and clays, and are less pronounced or even negligible in coarse-grained soils (Rollings and Rollings, 1996).

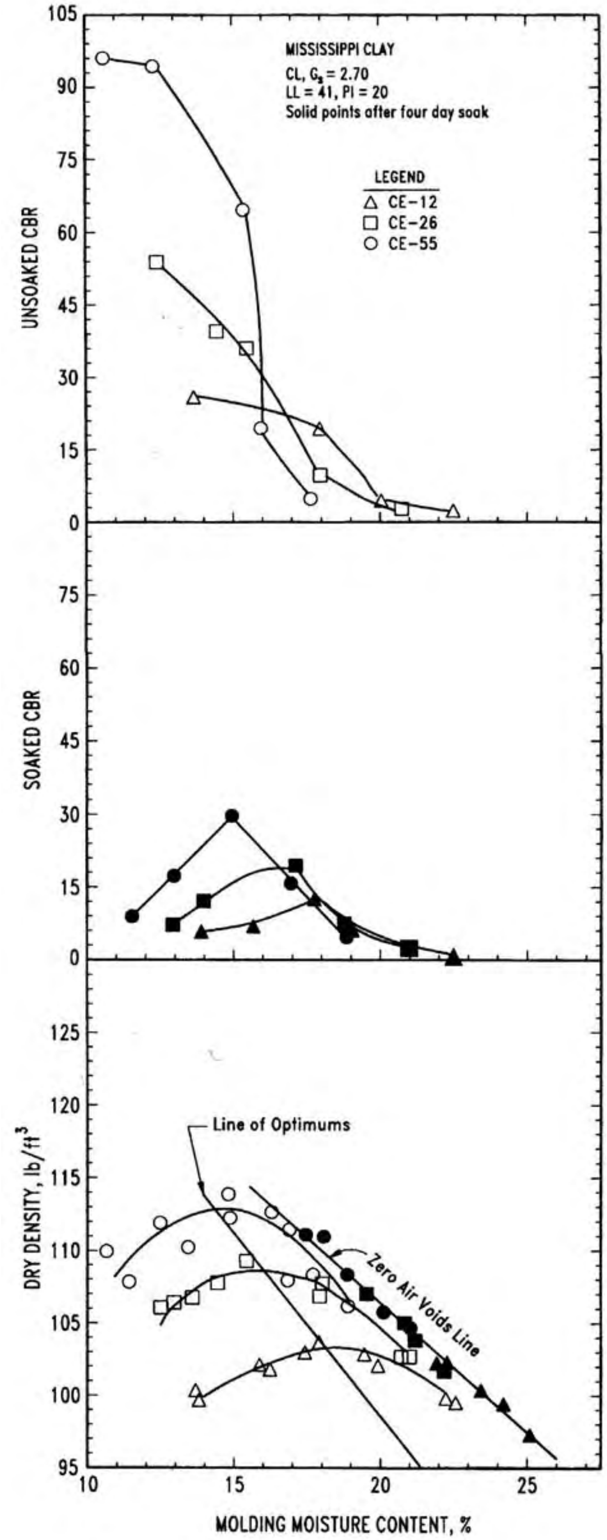


Figure 7-14. Strength as measured by CBR and dry density vs. water content for laboratory impact compaction (Rollings and Rollings, 1996).

- *Stiffness:* Figure 7-15 summarizes typical stiffness versus water content and compaction energy behavior for clays, where stiffness is defined as the stress required to case 5% and 25% axial strain in a triaxial compression test (Seed and Chan, 1959). Stiffness increases with compaction energy when compacted dry of optimum and is largely independent of compaction energy when compacted wet of optimum. The stiffness dry of optimum is also substantially larger than when compacted wet of optimum, as would be expected. **Again however, a significant decrease in stiffness can occur if the material becomes saturated to the extent that the stiffness could be less than that of the soil compacted wet of optimum.**
- *Permeability:* Permeability at constant compactive effort decreases with increasing water content and reaches a minimum at about the optimum moisture content. The permeability when compacted dry of optimum is about an order of magnitude higher than the value when compacted wet of optimum.
- *Swelling/Shrinkage Potential:* Swelling of compacted clays is greater when compacted dry of optimum. Dry clays have a greater capacity to absorb water, and thus swell more. Soils dry of optimum are in general more sensitive to environmental influences, such as changes in water content. The situation is just the opposite for shrinkage (Figure 7-16), where samples compacted wet of optimum exhibit the highest shrinkage strains as water is removed from the soil.

7.5 SUBGRADE CONDITIONS REQUIRING SPECIAL DESIGN ATTENTION

Considering variables such as soil type or mineralogy along a length of roadway, the geology (soil genesis and deposition method) and groundwater and flow properties make each project unique with respect to subgrade conditions. It is not surprising that certain conditions will exist that are not conducive to support, or even construction, of pavement systems. This section provides an overview of subgrade conditions that require special design attention. These subsurface conditions are often regional in nature and have usually been identified as problematic by the agency. Several foundation problems, such as collapsible or highly compressible soils, expansive or swelling soils, subsurface water and saturated soils, and frost-susceptible soils, occur extensively across the U.S. and are not specific to one region. For example, frost heave occurs in over half of the states in the U.S. and damage may be most severe in the central states, where many more frost cycles occur than in the most-northern states. Identification of these widely variable problematic subgrade conditions are also reviewed in this section, along with design and construction alternatives to achieve an adequate foundation on which to build the pavement structure.

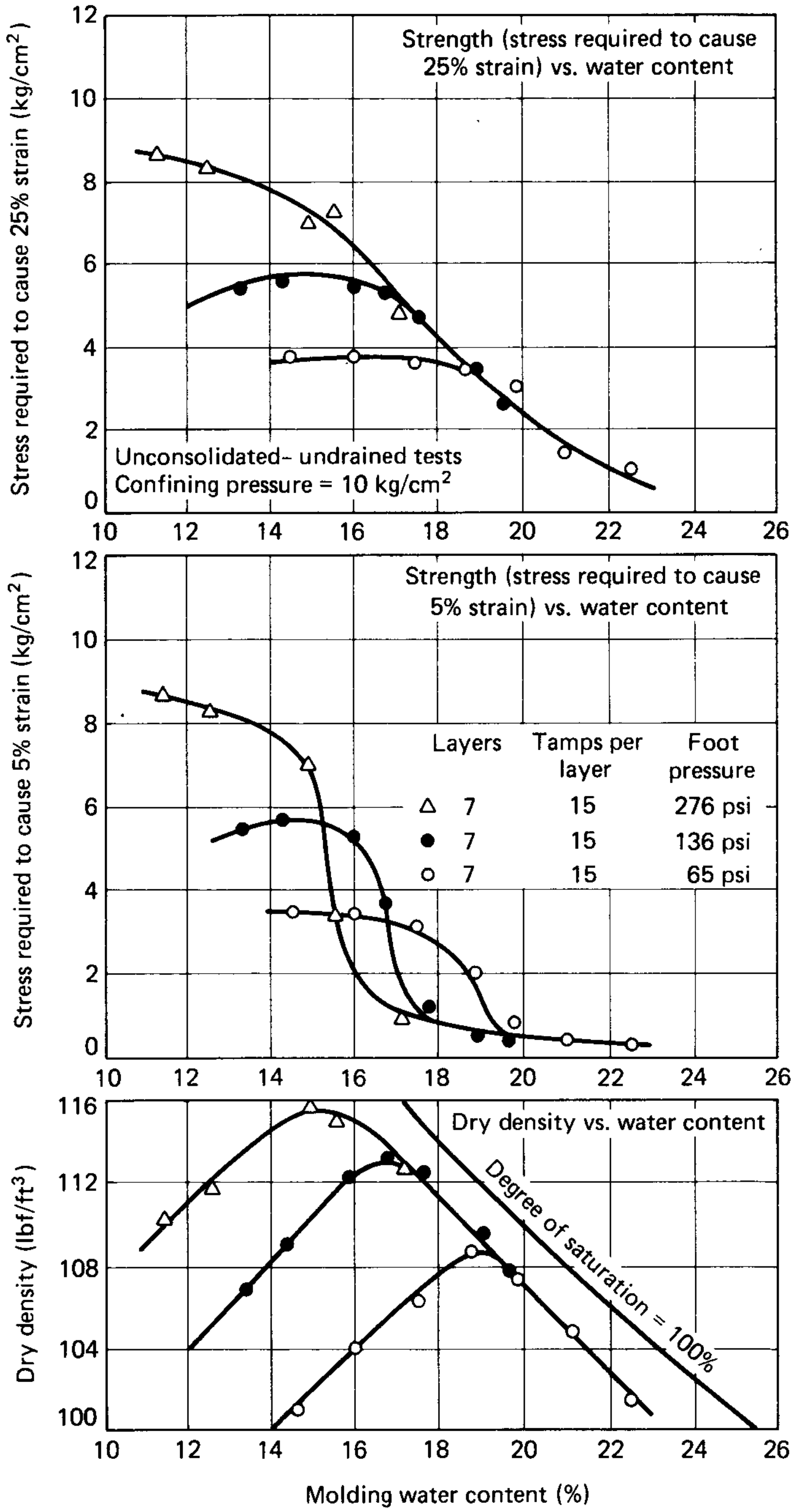


Figure 7-15. Stiffness as a function of compactive effort and water content (after Seed and Chan, 1959; from Holtz and Kovacs, 1981).

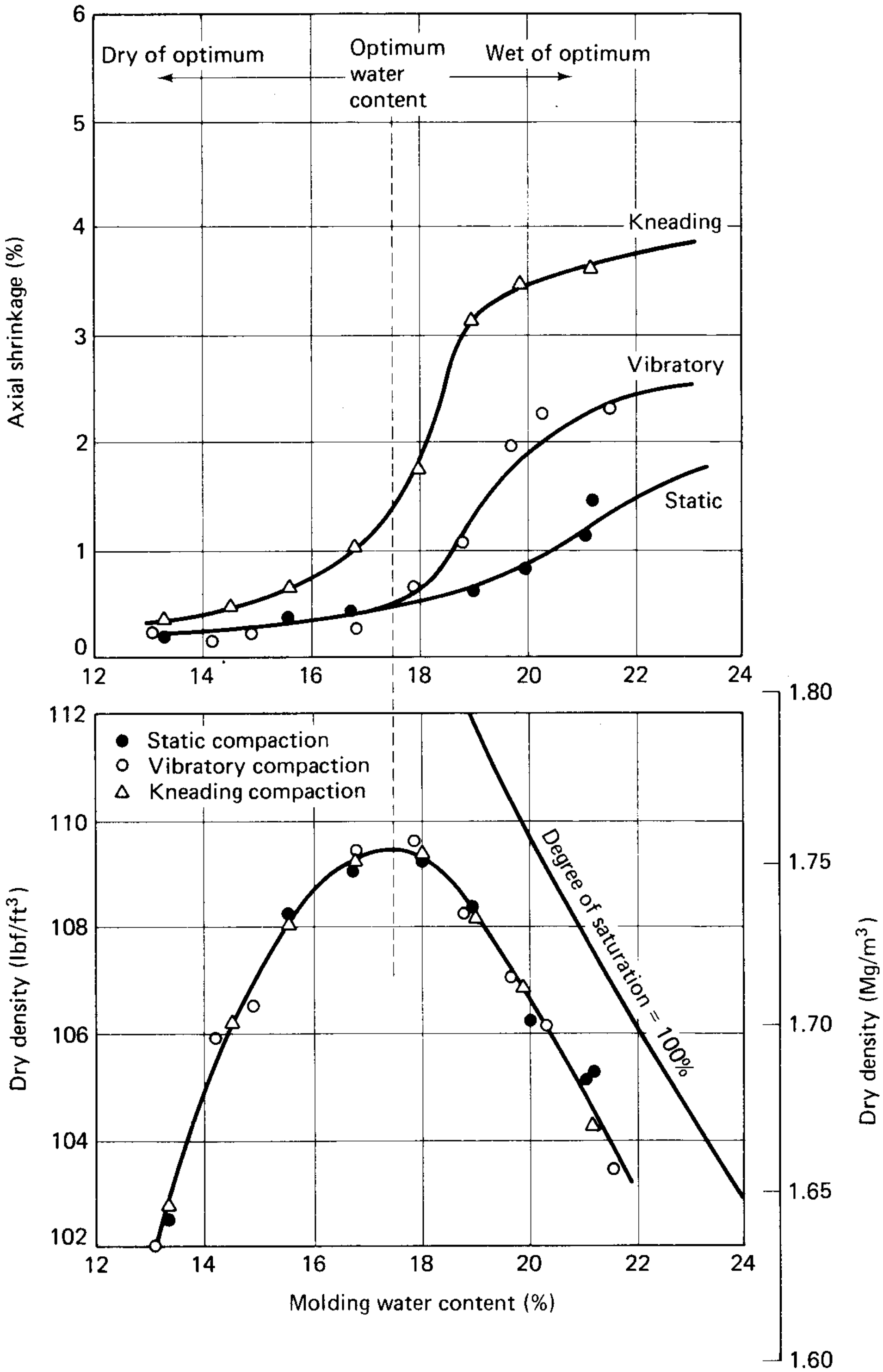


Figure 7-16. Shrinkage as a function of water content and type of compaction (after Seed and Chan, 1959; from Holtz and Kovacs, 1981).

Most of the subgrade conditions presented in this section can be anticipated through a complete exploration program, as described in Chapter 4, and mitigated or at least minimized via well-conceived designs. By identifying such subgrade issues in the design stage, or even the potential for such problems along an alignment, alternative designs can be established. Alternate designs can then be placed in the bid documents with indicators clearly identified that show where these alternatives should be considered, and then implemented if and where such conditions are encountered. When these special subgrade conditions are not recognized in design, they are often identified during construction, usually resulting in claims and overruns. However, identifying problems in construction is still somewhat fortunate, considering the impact such problems may have on the pavement performance. If the soil conditions described in this section go undetected, there typically is decreased serviceability, usually resulting in premature localized rehabilitation or, not uncommon, reconstruction of the pavement within the first few years of the pavement performance period.

7.5.1 Problematic Soil Types

Obviously, a pavement is to be constructed on whatever material and condition is naturally occurring. The strength and stability of some soils can present problems during construction and certainly can affect the long-term performance of the pavement during its service life. In order to properly discuss these potential problems, it is necessary to define some terms as they relate to problematic mineralogy (Sowers, 1979). Some of the terms are true geological terminology, while some are local or regional terminology. The terms may describe a particular material or condition, but all are problematic and care must be taken when constructing pavements in regions containing these materials.

Adobe. Sandy clays of medium plasticity found in the semiarid regions of the southwestern U.S. These soils have been used for centuries to make sun-dried brick. The name is also applied to some highly plastic clays of the West, which swell significantly when wet.

Bentonite. Highly plastic clay, usually montmorillonite, resulting from the decomposition of volcanic ash. It may be hard when dry, but swells considerably when wet.

Buckshot clay. Applied to clays of the southern and southwestern United States. Cracks into small, hard, relatively uniform sized lumps on drying. Dry lumps will degrade upon wetting (*e.g.,* after they have been used as fill). These soils also tend to swell when wet.

Caliche. A silt or sand of the semiarid areas of the southwestern United States that is cemented with calcium carbonate. The calcium carbonate is deposited by the evaporation of water brought to the ground surface by capillary action. The consistency of caliche varies from soft rock to firm soil.

Coquina. A soft, porous limestone made up largely of shells, coral, and fossils cemented together. Very friable, and breaks down during construction.

Gumbo. A fine-grained, highly plastic clay of the Mississippi Valley. It has a sticky, greasy feel, highly expansive, and forms large shrinkage cracks on drying.

Kaolin. A white or pink clay of low plasticity. It is composed largely of minerals of the kaolinite family.

Loam. A surface soil that may be described as a sandy silt of low plasticity or a silty sand that is well suited to tilling. It applies to soils within the uppermost horizons and should not be used to describe deep deposits of parent material. Loam-type soils are typically sensitive to moisture, easily disturbed in construction, and frost susceptible.

Loess. A deposit of relatively uniform, windblown silt. It has a loose structure, with numerous rootholes that produce vertical cleavage and high vertical permeability. It consists of angular to subrounded quartz and feldspar particles cemented with calcium carbonate or iron oxide. Upon saturation, it becomes soft and compressible because of the loss of cementing. Loess altered by weathering in a humid climate often becomes more dense and somewhat plastic (*loess loam*). Loess is also highly frost susceptible.

Marine clay. Clays deposited in a marine environment, which, if later uplifted, tend to be extra sensitive due to salt leaching, dramatically losing strength when disturbed.

Marl. A water-deposited sand, silt, or clay containing calcium carbonate. Marls are often light to dark gray or greenish in color and sometimes contain colloidal organic matter. They are often indurated into soft rock.

Muck or mud. An extremely soft, slimy silt or organic silt found on river and lake bottoms. The terms indicate an extremely soft consistency rather than any particular type of soil. Muck implies organic matter.

Peat. A naturally occuring highly organic substance derived primarily from plant materials (ASTM D 5715). Peats are dark brown or black, loose (void ratio may be 5 to 10), and extremely compressible. When dried, they will float. Peat bogs often emit quantities of inflammable methane gas. These soils will experience significant short-term and long-term settlement, even under light loads, and are often moisture sensitive, losing significant strength when wet. They are easily disturbed under construction activities. Peat containing a high degree of easily identifiable fibers is often called *fibrous peat* for geotechnical applications. Peat containing highly decomposed fibers and a significant highly organic soil component is often called *amorphous peat*.

Quicksand. Refers to a condition, not a soil type. Gravels, sands, and silts become "quick" when an upward flow of groundwater and/or gas takes place to such a degree that the particles are lifted.

Saprolites. Soils developed from in-situ weathering of rocks. Relic joints from the parent rock often control the weathered soils' strength, permeability, and stability. Fragments may appear sound, but prove to be weak. Identifying the transition of soil to weathered rock to sound rock is difficult, often resulting in claims.

Shale. Indurated, fine grained, sedimentary rocks, such as mudstones, siltstone, and claystone, which are highly variable and troublesome. Some are hard and stable, while others are soft and degrade into clay soon after exposure to the atmosphere or during the design life of the structure. Clays developed from shale are often highly plastic.

Sulfate. A mineral compound characterized by the sulfate radical SO_4, which may be contained in soil. It creates significant expansion problems in lime-stabilized soil and, in some cases, distress in concrete.

Sulfide. A mineral compound characterized by the linkage of sulfur with a metal, such as lead or iron, creating galena and pyrite, respectively.

Till. A mixture of sand, gravel, silt, and clay produced by the plowing action of glaciers. The name boulder clay is often given such soils, particularly in Canada and England. The characteristics of glacial till vary depending on the sediments and bedrock eroded. The tills in New England are typically coarser and less plastic that those from the Midwest. The tills in the Northeast tend to be broadly graded and often unstable under water action. The complex nature of their deposition creates a highly unpredictable material.

Topsoils. Surface soils that support plant life. They usually contain considerable organic matter. These soils tend to settle over time as organic matter continues to degrade. They are often moisture sensitive, losing significant strength when wet, and are easily disturbed under construction activities.

Tuff. The name applied to deposits of volcanic ash. In humid climates or in areas in which ash falls into bodies of water, the tuff becomes cemented into a soft, porous rock.

Varved clays. Sedimentary deposits consisting of alternate thin layers of silt and clay. Ordinarily, each pair of silt and clay layers is from 3 – 13 mm (1/8 – 1/2 in.) thick. They are the result of deposition in lakes during periods of alternating high and low water in the inflowing streams, and are often formed in glacial lakes. These deposits have a much higher horizontal than vertical permeability, with the horizontal seams holding water. They are often sensitive, and will lose strength when remolded.

7.5.2 Compressible Soils

Effect of Compressible Soils on Pavement Performance

Highly compressible (very weak) soils are susceptible to large settlements and deformations with time that can have a detrimental effect on pavement performance. Highly compressible soils are very low density, saturated soils, usually silts, clays, and organic alluvium or wind blown deposits and peats. If these compressible soils are not treated properly, large surface depressions with random cracking can develop. The surface depressions can allow water to pond on the pavement's surface and more readily infiltrate the pavement structure, compounding a severe problem. More importantly, the ponding of water will create a safety hazard to the traveling public during wet weather.

Treatments for Compressible Soils

The selection of a particular technique depends on the depth of the weak soil, and the difference between the in-situ conditions and the minimum compaction or strength requirements to limit the amount of anticipated settlement to a permissible value that will not adversely affect pavement performance. When constructing roadways in areas with deep deposits of highly compressible layers, the specific soil properties must be examined to calculate the estimated settlement. Under these conditions, a geotechnical investigation and detailed settlement analysis must be completed prior to the pavement design. When existing subgrade soils do not meet minimum compaction requirements and are susceptible to large settlements over time, consider the following alternatives:

- Remove and process soil to attain the approximate optimum moisture content, and replace and compact.

- Remove and replace subgrade soil with suitable borrow or select embankment materials. All granular fill materials should be compacted to at least 95% of the maximum density, with moisture control, as defined by AASHTO T180. Cohesive fill materials should be compacted to no less than 90%, near or slightly greater than optimum moisture content (*e.g.,* -1% to +2% of optimum), as defined by AASHTO T99.
- Consider mechanical stabilization using geosynthetics as covered in Section 7.5 to reduce the amount of undercut required.
- If soils are granular (*e.g.,* sands and some silts), consider compaction of the soils from the surface to increase the dry density through dynamic compaction techniques. Identification of soil characteristics and detailed procedures for the successful implementation of this technique covered in FHWA/NHI course 132034 on *Ground Improvement Techniques* (FHWA NHI-04-001).
- If the soil is extremely wet or saturated, consider dewatering using well points or deep horizontal drains. If horizontal drains cannot be daylighted, connection to storm drainage pipes or sump pumps may be required.
- Consolidate deep deposits of very weak saturated soils with large fills prior to pavement construction (surcharge). After construction, the fills can either be left in-place or removed, depending on the final elevation. Consider wick drains to accelerate consolidation (see FHWA NHI-04-001).
- Other techniques for deep deposits of compressible soil include piled embankments and use of lightweight fill, such as geofoam, as covered in the FHWA *Ground Improvement Techniques* manual (FHWA NHI-04-001). Although more costly than most of the previous techniques in terms of construction dollars, these techniques offer immediate improvement, thus accelerating construction. On some projects, the time savings may be more valuable than the construction cost differential.

7.5.3 Collapsible Soils

As with highly compressible soils, collapsible soils can lead to significant localized subsidence of the pavement. Collapsible soils are very low density silt type soils, usually alluvium or wind blown (loess) deposits, and are susceptible to sudden decreases in volume when wetted. Often their unstable structure has been cemented by clay binders or other deposits, which will dissolve on saturation, allowing a dramatic decrease in volume (Rollings and Rollings, 1996). Native subgrades of collapsible soils should be soaked with water prior to construction and rolled with heavy compaction equipment. In some cases, residual soils may also be collapsible due to leaching of colloidal and soluble materials. Figure 7-17 provides a method of identifying the potential for collapsible soils. Other local methods for identification may be available. Collapsible soils can also be created in fills when sand type soils are compacted on the dry side of optimum moisture. Meniscus forces between particles can create a soil fabric susceptible to collapse.

If pavement systems are to be constructed over collapsible soils, special remedial measures may be required to prevent large-scale cracking and differential settlement. To avoid problems, collapse must be induced prior to construction. Methods include

1. ponding water over the region of collapsible soils.
2. infiltration wells.
3. compaction - conventional with heavy vibratory roller for shallow depths (within 0.3 or 0.6 m (1 or 2 ft))
4. compaction - dynamic or vibratory for deeper deposits of more than half a meter (a few feet) (could be combined with inundation)
5. excavated and replaced.

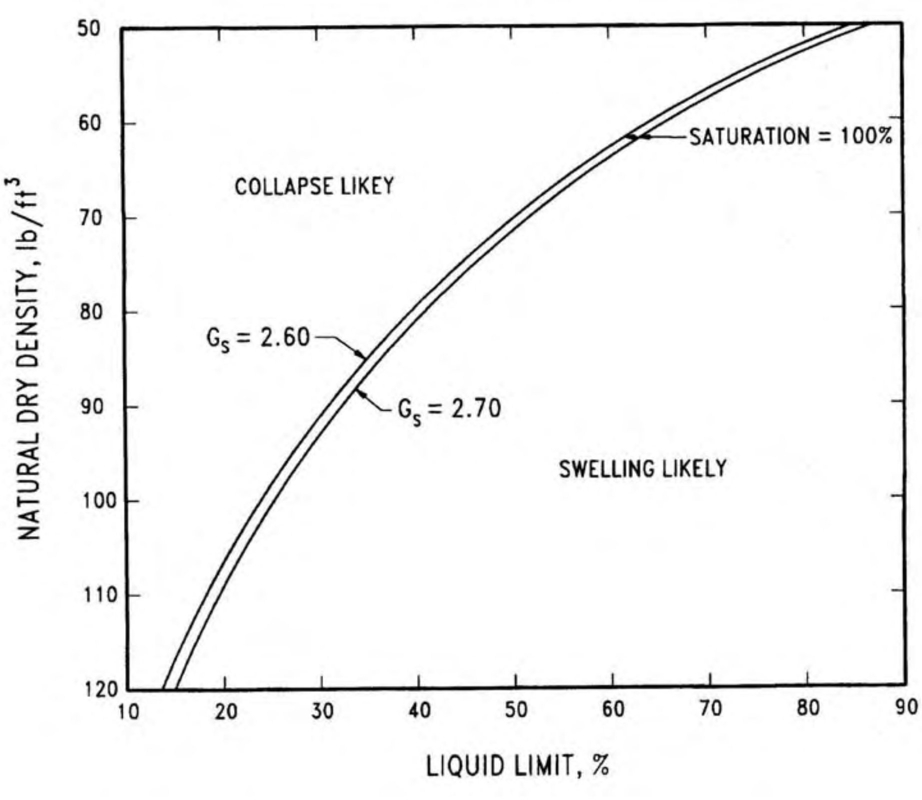


Figure 7-17. Guide to collapsible soil behavior (Rollings and Rollings, 1996).

7.5.4 Swelling Soils

Effect of Swelling Soils on Pavement Performance

Swelling or expansive soils are susceptible to volume change (shrink and swell) with seasonal fluctuations in moisture content. The magnitude of this volume change is dependent on the type of soil (shrink-swell potential) and its change in moisture content. A loss of moisture will cause the soil to shrink, while an increase in moisture will cause it to expand or swell. This volume change of clay type soils can result in longitudinal cracks near the pavement's edge and significant surface roughness (varying swells and depressions) along the pavement's length.

Expansive soils are a very significant problem in many parts of the United States (see Figure 7-18) and are responsible for the application of premature maintenance and rehabilitation activities on many miles of roadway each year. Expansive soils are especially a problem when deep cuts are made in a dense (over-consolidated) clay soil.

Identification of Swelling Soils

Various techniques and procedures exist for identifying potentially expansive soils. AASHTO T 258 can be used to identify soils and conditions that are susceptible to swell. Two of the more commonly used documents are listed below:

- *An Evaluation of Expedient Methodology for Identification of Potentially Expansive Soils*, Report No. FHWA-RD-77-94, Federal Highway Administration, Washington, D.C., June 1977.
- *Design and Construction of Airport Pavements on Expansive Soils*, Report No. FAA-RD-76-66, Federal Aviation Administration, U.S. Department of Transportation, Washington, D.C., June 1976.

Clay mineralogy and the availability of water are the key factors in determining the degree to which a swelling problem may exist at a given site. Different clay minerals exhibit greater or lesser degrees of swell potential based on their specific chemistry. Montmorillonitic clays tend to exhibit very high swell potentials due to the particle chemistry, whereas illitic clays tend to exhibit very low swell potentials. Identification of clay minerals through chemical or microscopic means may be used as a method of identifying the presence of high swell potential in soils. The soil fabric will also influence the swell potential, as aggregated particles will tend to exhibit higher swell than dispersed particles, and flocculated higher than deflocculated. Generally, the finer-grained and more plastic the soil, the higher the swell potential the soil will exhibit.

The identification of swelling soils in the subgrade is a key component of the geotechnical investigation for the roadway. Soils at shallow depths beneath the proposed pavement elevation are generally sampled as part of the investigation, and their swell potential may be identified in a number of ways. Index testing is a common method for identifying swell potential. Laboratory testing to obtain the plastic and liquid limits and/or the shrinkage limit will usually be conducted. The soil activity (ASTM D 4318), defined as the ratio of the plasticity index to the percentage of the soil by weight finer than 0.002 mm (0.08 mils) is also used as an index property for swell potential, since clay minerals of higher activity exhibit higher swell. Activity calculation requires measurement of gradation using hydrometer methods, which is not typical in geotechnical investigations for pavement design in many states. In addition to index testing, agency practice in regions where swelling soils are a common problem may include swell testing (e.g., ASTM D 4546), for natural or compacted soil samples. Such testing generally includes measurement of the change in height (or volume) of a sample exposed to light loading similar to that expected in the field and then allowed free access to water.

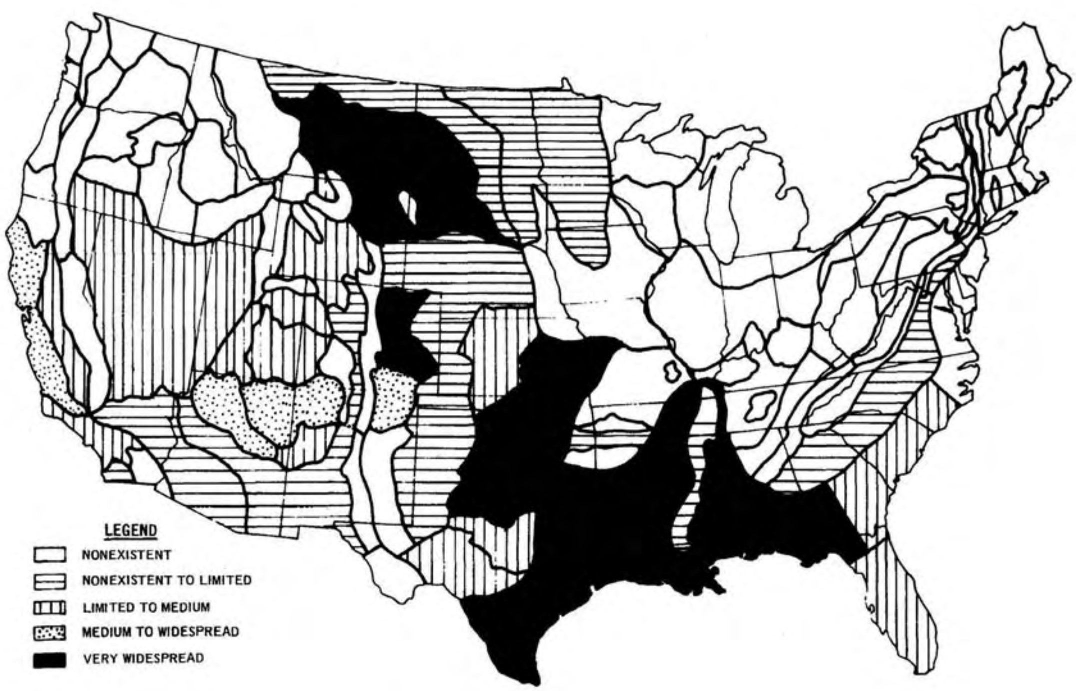


Figure 7-18. Estimated location of swelling soils (from Witczak, 1972).

Treatment for Swelling Soils

When expansive soils are encountered along a project in environments and areas where significant moisture fluctuations in the subgrade are expected, consideration should be given to the following alternatives to minimize future volume change potential of the expansive soil:

- For relatively thin layers of expansive clays near the surface, remove and replace the expansive soil with select borrow materials.
- Extend the width of the subsurface pavement layers to reduce the change (*i.e.,* wetting or drying) in subgrade moisture along the pavement's edge, and increase the roadway crown to reduce infiltration moisture.
- Partial encapsulation along the edge of the pavement or full encapsulation can also be used to reduce change in subgrade moisture, as described in greater detail in Section 7.5.
- Scarify, stabilize, and recompact the upper portion of the expansive clay subgrade. Lime or cement stabilization is an accepted method for controlling the swelling of soils, as discussed in Section 7.6. (*Stabilization*, as used for expansive soils, refers to the treatment of a soil with such agents as bitumen, Portland cement, slaked or hydrated lime, and flyash to limit its volume change characteristics. This can substantially increase the strength of the treated material.)
- In areas with deep cuts in dense, over-consolidated expansive clays, complete the excavation of the subsurface soils to the proper elevation, and allow the subsurface soils to rebound prior to placing the pavement layers.

AASHTO 1993 (Appendix C) provides procedures and graphs to predict the direct effect of swelling soils on serviceability loss and is treated with respect to the differential effects on the longitudinal profile of the road surface. If the swelling is anticipated to be relatively uniform, then the procedures do not apply.

7.5.5 Subsurface Water

It is important to identify any saturated soil strata, the depth to groundwater, and subsurface water flow between soil strata. Subsurface water is especially important to recognize and identify in the transition areas between cut and fill segments. If allowed to saturate unbound base/subbase materials and subgrade soils, subsurface water can significantly decrease the strength and stiffness of these materials. Reductions in strength can result in premature surface depressions, rutting, or cracking. Seasonal moisture flow through selected soil strata can also significantly magnify the effects of differential volume change in expansive soils. Cut areas are particularly critical for subsurface water.

Treatments for Subsurface Water

When saturated soils or subsurface water are encountered, consideration should be given to the following alternatives for improving the foundation or supporting subgrade:

- For saturated soils near the surface, dry or strengthen the wet soils through the use of mechanical stabilization techniques to provide a construction platform for the pavement structure, as described in Section 7.6.
- Remove and replace the saturated soils with select borrow materials or soils. (May not be an option if excavation is required below the groundwater level).
- Place and properly compact thick fills or embankments to increase the elevation of the subgrade, or in other words, increase the thickness between the saturated soils or water table depth and pavement structure.
- Consideration should also be given to the use of subgrade drains as previously detailed in Section 7.2 whenever the following conditions exist:
  - High ground-water levels that may reduce subgrade stability and provide a source of water for frost action.
  - Subgrade soils consisting of silts and very fine sands that may become quick or spongy when saturated.
  - Water seeps from underlying water-bearing strata or from subgrades in cut areas (consider intercepting drains).

7.5.6 Frost-Susceptible Soils

Effect of Frost Action on Pavement Performance

Frost action can cause differential heaving, surface roughness and cracking, blocked drainage, and a reduction in bearing capacity during thaw periods. These effects range from slight to severe, depending on types and uniformity of subsoil, regional climatic conditions (*i.e.,* depth of freeze), and the availability of water.

One effect of frost action on pavements is frost heaving caused by crystallization of ice lenses in voids of soils containing fine particles. As shown in Figure 7-19, three conditions must be present to cause frost heaving and associated frost action problems:

- frost-susceptible soils;
- subfreezing temperatures in the soil; and,
- source of water.

If these conditions occur uniformly, heaving will be uniform; otherwise, differential heaving will occur, causing surface irregularities, roughness, and ultimately cracking of the pavement surface.

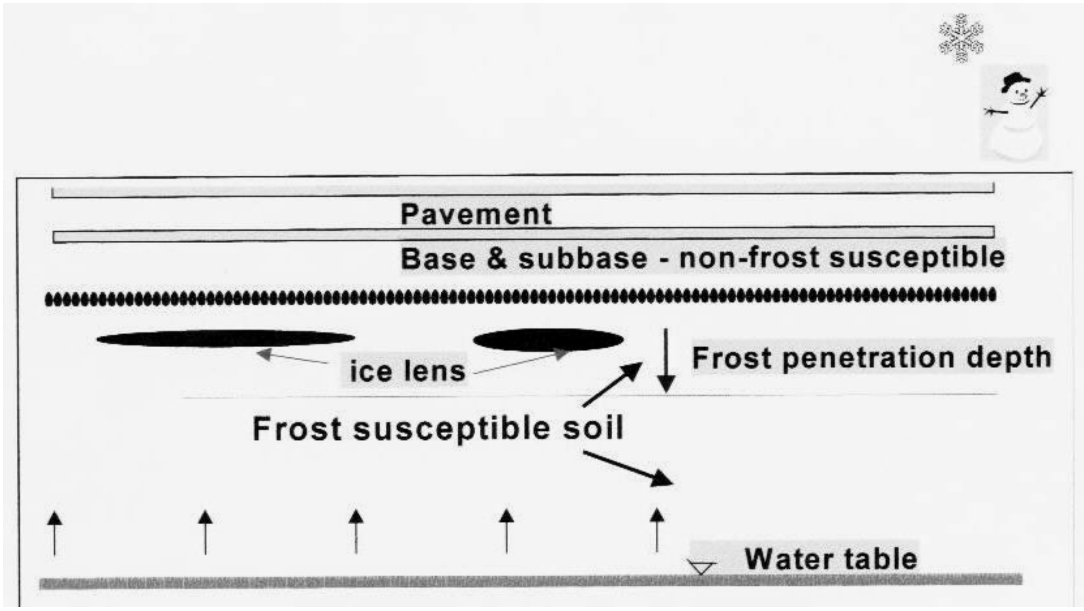


Figure 7-19. Elements of frost heave.

A second effect of frost action is thaw weakening. The bearing capacity may be reduced substantially during mid-winter thawing periods, and subsequent frost heaving is usually more severe because water is more readily available to the freezing zone. In more-southerly areas of the frost zone, several cycles of freeze and thaw may occur during a winter season and cause more damage than one longer period of freezing in more-northerly areas. Spring thaws normally produce a loss of bearing capacity to well below summer and fall values, followed by a gradual recovery over a period of weeks or months. Water is also often trapped above frozen soil during the thaw, which occurs from the top down, creating the potential for long-term saturated conditions in pavement layers.

Identification of Frost-Susceptible Soils

Frost-susceptible soils have been classified into four general groups. Table 7-12 provides a summary of the typical soils in each of these four groups based on the amount of fines (material passing the 0.075 mm (No. 200) sieve. Figure 7-20 graphically displays the expected average rate of frost heave for the different soil groups based on portion of soil finer than 0.02 mm (0.8 mils).

Little to no frost action occurs in clean, free draining sands, gravels, crushed rock, and similar granular materials, under normal freezing conditions. The large void space permits water to freeze in-place without segregation into ice lenses. Conversely, silts are highly frost-susceptible. The condition of relatively small voids, high capillary potential/action, and relatively good permeability of these soils accounts for this characteristic.

Table 7-12. Frost susceptibility classification of soils (NCHRP 1-37A).

Frost Group	Degree of Frost Susceptibility	Type of Soil	Percentage Finer than 0.075 mm (# 200) by wt.	Typical Soil Classification
F1	Negligible to low	Gravelly soils	3-10	GC, GP, GC-GM, GP-GM
F2	Low to medium	Gravelly soils	10-20	GM, GC-GM, GP-GM
		Sands	3-15	SW, SP, SM, SW-SM, SP-SM
F3	High	Gravelly Soils	Greater than 20	GM-GC
		Sands, except very fine silty sands	Greater than 15	SM, SC
		Clays PI>12	—	CL, CH
F4	Very high	All Silts	—	ML-MH
		Very Fine Silty Sands	Greater than 15	SM
		Clays PI<12	—	CL, CL-ML
		Varied clays and other fine grained, banded sediments	—	CL, ML, SM, CH

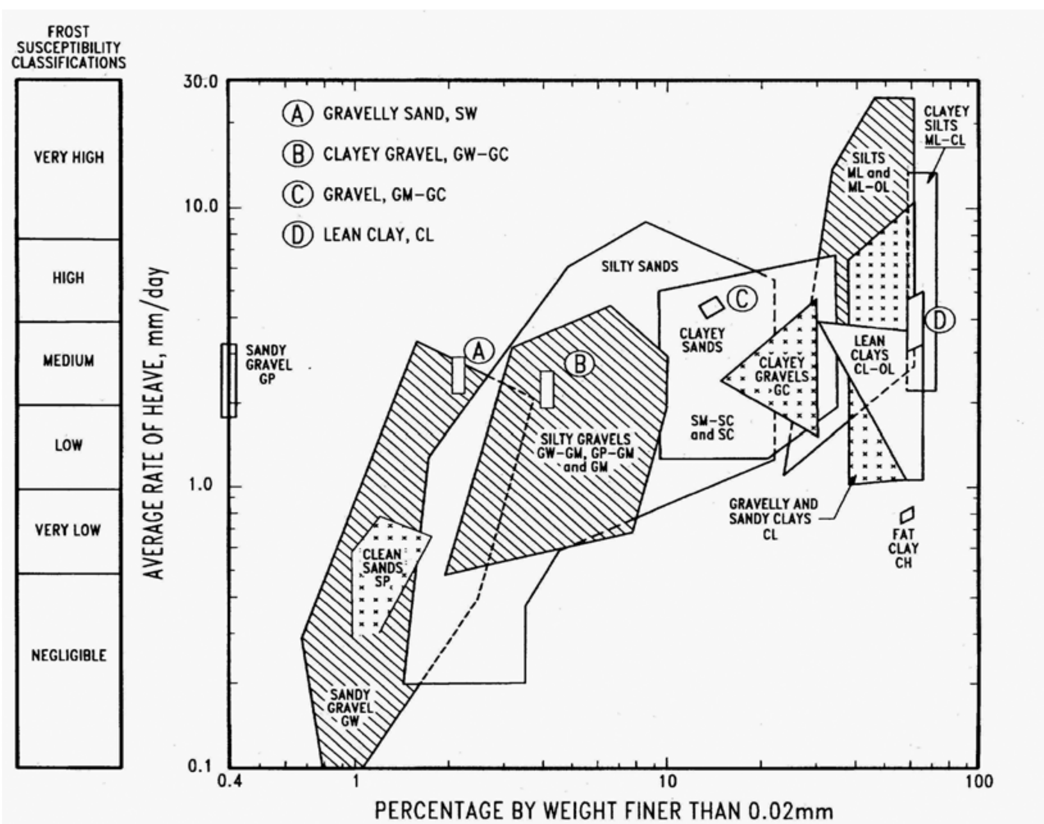


Figure 7-20. Average rate of heave versus % fines for natural soil gradations (Kaplar, 1974).

Clays are cohesive and, although their potential capillary action is high, their capillary rate is low. Although frost heaving can occur in clay soils, it is not as severe as for silts, since the impervious nature of the clays makes passage of water slow. The supporting capacity of clays must be reduced greatly during thaws, even in the absence of significant heave. Thawing usually takes place from the top downward, leading to very high moisture contents in the upper strata.

A groundwater level within 1.5 m (5 ft) of the proposed subgrade elevation is an indication that sufficient water will exist for ice formation. Homogeneous clay subgrade soils also contain sufficient moisture for ice formation, even with depth to groundwater in excess of 3 m (10 ft). However, the magnitude of influence will be highly dependent on the depth of the freezing front (*i.e.,* frost depth penetration). For deep frost penetration, groundwater at even a greater depth could have an influence on heave.

Identification of Frost-Susceptible Conditions

The most distinguishing factor for identifying a pavement frost hazard condition is water supply. For frost susceptible soils within the frost zone, the frost hazard may be rated as high or low, according to the following conditions. An unknown rating may be appropriate when conditions for both high and low ratings occur and cannot be resolved, or when little or no information is available. The inclusion of a frost hazard rating in the site evaluation documentation verifies that an evaluation of frost action has been attempted and has not been overlooked. When the rating is unknown, a decision to include frost action mitigation measures in a design will be based more upon the unacceptable nature of frost damage than the probability of occurrence.

The conditions associated with a high frost hazard potential include

1. A water table within 3 m (10 ft) of the pavement surface (depth of influence depends on the type of soil and frost depth).
2. Observed frost heaves in the area.
3. Inorganic soils containing more than 3% (by weight) or more grains finer than 0.02 mm (0.8 mils) in diameter according to the U.S. Army Corps of Engineers.
4. A potential for the ponding of surface water and the occurrence of soils between the frost zone beneath the pavement and the surface water with permeabilities high enough to enable seepage to saturate soils within the frost zone during the term of ponding.

The conditions associated with a low frost hazard potential include

1. A water table greater than 6 m (20 ft) below the pavement surface (again, could be much shallower depending on the type of soil and frost depth).

2. Natural moisture content in the frost zone low versus the saturation level.
3. Seepage barriers between the water supply and the frost zone.
4. Existing pavements or sidewalks in the vicinity with similar soil and water supply conditions and without constructed frost protection measures that have not experienced frost damage.
5. Pavements on embankments with surfaces more than 1 – 2 m (3 – 6 ft) above the adjacent grades (provides some insulation and a weighting action to resist heave).

Treatment for Frost Action

When frost-susceptible soils are encountered, consideration should be given to the following alternatives for improving the foundation or supporting subgrade:

1. Remove the frost-susceptible soil (generally for groups F3 and F4, Table 7-12) and replace with select non-frost susceptible borrow to the expected frost depth penetration.
2. Place and compact select non-frost-susceptible borrow materials to a thickness or depth to prevent subgrade freezing for frost susceptible soil groups F2, F3, and F4, Table 7-12.
3. Remove isolated pockets of frost-susceptible soils to eliminate abrupt changes in subgrade conditions.
4. Stabilize the frost-susceptible soil by eliminating the effects of soil fines by three processes: a) mechanically removing or immobilizing by means of physical-chemical means, such as cementitious bonding, b) effectively reducing the quantity of soil moisture available for migration to the freezing plane, as by essentially blocking off all migratory passages, or c) altering the freezing point of the soil moisture.
    a. Cementing agents, such as Portland cement, bitumen, lime, and lime-flyash, as covered in Section 7.5. These agents effectively remove individual soil particles by bonding them together, and also act to partially remove capillary passages, thereby reducing the potential for moisture movement. Care must be taken when using lime and lime-flyash mixtures with clay soils in seasonal frost areas (see Section 7.5 & Appendix F).
    b. Soil moisture available for frost heave can be mitigated through the installation of deep drains and/or a capillary barrier such that the water table is maintained at a sufficient depth to prevent moisture rise in the freezing zone. Capillary barriers can consist of either an open graded gravel layer sandwiched between two geotextiles, or a horizontal geocomposite drain. The installation of a capillary barrier requires the removal of the frost susceptible material to a depth either below frost penetration or sufficiently significant to reduce the influence of frost heave on the pavement. The capillary break must

be drained. The frost susceptible soil can then be replaced and compacted above the capillary barrier to the required subgrade elevation.

5. Increase the pavement structural layer thickness to account for strength reduction in the subgrade during the spring-thaw period for frost-susceptible groups F1, F2, and F3.

Pavement design for frost action often determines the required overall thickness of flexible pavements and the need for additional select material beneath both rigid and flexible pavements. Three design approaches have been used for pavement in seasonal frost areas:

- The Complete Protection approach—requires non-frost susceptible materials for the entire depth of frost (*e.g.,* treatment methods 1, 2, and 3 above).

- Limited Subgrade Frost Penetration approach—permits some frost penetration into the subgrade, but not enough to allow unacceptable surface roughness to develop.

- Reduced Subgrade Strength approach—allows more frost penetration into the subgrade, but provides adequate strength during thaw weakened periods.

AASHTO 1993 (Appendix C) provides procedures and graphs to predict the direct effect of frost heave on serviceability loss and is treated with respect to the differential effects on the longitudinal profile of the road surface. If the frost is anticipated to be relatively uniform, then the procedures do not apply.

For the most part, local frost-resistant design approaches have been developed from experience, rather than by application of some rigorous theoretical computational method. A more rigorous method is available in the NCHRP 1-37A design procedure to reduce the effects of seasonal freezing and thawing to acceptable limits, as discussed in Chapter 6. The Enhanced Integrated Climatic Model is used to determine the maximum frost depth for the pavement system at a particular location. Various combinations of layer thicknesses and material types can be evaluated in terms of their impact on the maximum frost depth and total amount of base and select materials necessary to protect the frost susceptible soils from freezing.

7.5.7 Summary

Problematic soils can be treated using a variety of methods or a combination thereof. Improvement techniques that can be used to improve the strength and reduce the climatic variation of the foundation on pavement performance include

1. Improvement of subsurface drainage (see Section 7.2, and should always be considered).
2. Removal and replacement with better materials (*e.g.,* thick granular layers).
3. Mechanical stabilization using thick granular layers.
4. Mechanical stabilization of weak soils with geosynthetics (geotextiles and geogrids) in conjunction with granular layers.
5. Lightweight fill.
6. Stabilization of weak soils with admixtures (highly plastic or compressible soils).
7. Soil encapsulation.

Details for most of these stabilization methods will be reviewed in the next section.

7.6 SUBGRADE IMPROVEMENT AND STRENGTHENING

Proper treatment of problem soil conditions and the preparation of the foundation are extremely important to ensure a long-lasting pavement structure that does not require excessive maintenance. Some agencies have recognized certain materials simply do not perform well, and prefer to remove and replace such soils (*e.g.,* a state specification dictating that frost susceptible loess cannot be present in the frost penetration zone). However, in many cases, this is not the most economical or even desirable treatment (*e.g.,* excavation may create disturbance, plus additional problems of removal and disposal). Stabilization provides an alternate method to improve the structural support of the foundation for many of the subgrade conditions presented in the previous section. In all cases, the provision for a uniform soil relative to textural classification, moisture, and density in the upper portion of the subgrade cannot be over-emphasized. This uniformity can be achieved through soil sub-cutting or other stabilization techniques. Stabilization may also be used to improve soil workability, provide a weather resistant work platform, reduce swelling of expansive materials, and mitigate problems associated with frost heave. In this section, alternate stabilization methods will be reviewed, and guidance will be presented for the selection of the most appropriate method.

7.6.1 Objectives of Soil Stabilization

Soils that are highly susceptible to volume and strength changes can cause severe roughness and accelerate the deterioration of the pavement structure in the form of increased cracking and decreased ride quality when combined with truck traffic. Generally, the stiffness (in terms of resilient modulus) of some soils is highly dependent on moisture and stress state (see Section 5.4). In some cases, the subgrade soil can be treated with various materials to improve the strength and stiffness characteristics of the soil. Stabilization of soils is usually performed for three reasons:

1. As a construction platform to dry very wet soils and facilitate compaction of the upper layers—for this case, the *stabilized* soil is usually not considered as a structural layer in the pavement design process.
2. To strengthen a weak soil and restrict the volume change potential of a highly plastic or compressible soil—for this case, the *modified* soil is usually given some structural value or credit in the pavement design process.
3. To reduce moisture susceptibility of fine grain soils.

A summary of the stabilization methods most commonly used in pavements, the types of soils for which they are most appropriate, and their intended effects on soil properties is presented in Table 7-13.

Mechanical stabilization using thick gravel layers or granular layers in conjunction with geotextiles or geogrids is an effective technique for improving roadway support over soft, wet subgrades. Thick granular layers provide a working platform, but do not provide strengthening of the subgrade. In fact, construction of thick granular layers in some cases results in disturbance of the subgrade due to required construction activities. Thick granular layers are also used to avoid or reduce frost problems by providing a protection to the underlying subgrade layers.

Table 7-13. Stabilization Methods for Pavements (after Rollings and Rollings, 1996).

Stabilization Method	Soil Type	Improvement	Remarks
Mechanical			
- More Gravel	Silts and Clays	None	Reduce dynamic stress level
- Blending	Moderately plastic	None	Too difficult to mix
	Other	Improve gradation Reduce plasticity Reduce breakage	
- Geosynthetics	Silts and Clays	Strength gain through minimum disturbance and consolidation	Fast, plus provides long-term separation
- Lightweight fill	Very weak silts, clays, peats	None Thermal barrier for frost protection	Fast, and reduces dynamic stress level
Admixture			
- Portland cement	Plastic		Less pronounced hydration of cement
	Coarse		Hydration of cement
- Lime	Plastic	Drying	Rapid
		Strength gain	Rapid
		Reduce plasticity	Rapid
		Coarsen texture	Rapid
		Long-term pozzolanic cementing	Slow
	Coarse with fines	Same as plastic	Dependent on quantity of plastic fines
	Nonplastic	None	No reactive material
- Lime-flyash	Same as lime	Same as lime	Covers broader range
- Lime-cement- flyash	Same as lime	Same as lime	Covers broader range
- Bituminous	Coarse	Strengthen/bind waterproof	Asphalt cement or liquid asphalt
	Some fines	Same as coarse	Liquid asphalt
	Fine	None	Can't mix
- Pozzolanic and slags	Silts and coarse	Acts as a filler	Dense and strong
		Cementing of grains	Slower than cement
- Chemicals	Plastic	Strength increase and volume stability	See vendor literature Difficult to mix
Water proofers			
- Asphalt	Plastic and collapsible	Reduce change in moisture	Long-term moisture migration problem
- Geomembranes	Plastic and collapsible	Reduce change in moisture	Long-term moisture migration problem

A common practice in several New England and Northwestern states is to use a meter (3.3 ft) or more of gravel beneath the pavement section. The gravel improves drainage of surface infiltration water and provides a weighting action that reduces and results in more uniform heave. Washington State recently reported the successful use of an 0.4 m (18 in.) layer of cap rock beneath the pavement section in severe frost regions (Ulmeyer et al., 2002).

Blending gravel and, more recently, recycled pavement material with poorer quality soils also can provide a working platform. The gravel acts as filler, creating a dryer condition and decreasing the influence of plasticity. However, if saturation conditions return, the gravel blend can take on the same poorer support characteristics of the subgrade.

Geotextiles and geogrids used in combination with quality aggregate minimize disturbance and allow construction equipment access to sites where the soils are normally too weak to support the initial construction work. They also allow compaction of initial lifts on sites where the use of ordinary compaction equipment is very difficult or even impossible. Geotextiles and geogrids reduce the extent of stress on the subgrade and prevent base aggregate from penetrating into the subgrade, thus reducing the thickness of aggregate required to stabilize the subgrade. Geotextiles also act as a separator to prevent subgrade fines from pumping or otherwise migrating up into the base. Geosynthetics have been found to allow for subgrade strength gain over time. However, the primary long-term benefit is preventing aggregate-subgrade mixing, thus maintaining the thickness of the base and subbase. In turn, rehabilitation of the pavement section should only require maintenance of surface pavement layers.

Stabilization with admixtures, such as lime, cement, and asphalt, have been mixed with subgrade soils used for controlling the swelling and frost heave of soils and improving the strength characteristics of unsuitable soils. For admixture stabilization or modification of cohesive soils, hydrated lime is the most widely used. Lime is applicable in clay soils (CH and CL type soils) and in granular soils containing clay binder (GC and SC), while Portland cement is more commonly used in non-plastic soils. Lime reduces the Plasticity Index (PI) and renders a clay soil less sensitive to moisture changes. The use of lime should be considered whenever the PI of the soil is greater than 12. Lime stabilization is used in many areas of the U.S. to obtain a good construction platform in wet weather above highly plastic clays and other fine-grained soils. It is important to note that changing the physical properties of a soil through chemical stabilization can produce a soil that is susceptible to frost heave. Following is a brief description of the characteristics of stabilized soils followed by the treatment procedures. Additional guidance on soil stabilization with admixtures and stabilization with geosynthetics can be obtained from the following resources:

- *"*Lime Stabilization – Reactions, Properties, Design, and Construction," *State of the Art Report 5*, Transportation Research Board, Washington D.C., 1987.
- *Soil Stabilization for Pavements,* Joint Departments of the Army and Air Force, USA, TM 5-822-14/AFMAN 32-8010, 1994.
- *Geosynthetics Design and Construction Guidelines*, FHWA HI-95-038, 1998.
- *Standard Specifications for Geotextiles* - AASHTO M288, 1997.

7.6.2 Characteristics of Stabilized Soils

Although mechanical stabilization with thick granular layers or geosythetics and aggregate subbase provides the potential for strength improvement of the subgrade over time, this is generally not considered in the design of the pavement section, and no increase in structural support is attributed to the geosynthetic. However, the increase in gravel thickness (minus an allowance for rutting) can contribute to the support of the pavement. Alternatively, the aggregate thickness used in conjunction with the geosynthetic is designed to provide an equivalent subgrade modulus, which can be considered in the pavement design, discounting the additional aggregate thickness of the stabilization layer. Geosynthetics also allow more open graded aggregate, thus providing for the potential to drain the subbase into edgedrains and improving its support value.

The improvement of subgrade or unbound aggregate by application of a stabilizing agent is intended to cause the improvements outlined above (*i.e.,* construction platform, subgrade strengthening, and control of moisture). These improvements arise from several important mechanisms that must be considered and understood by the pavement designer. Admixtures used as subgrade stabilizing agents may fill or partially fill the voids between the soil particles. This reduces the permeability of the soil by increasing the tortuosity of the pathways for water to migrate through the soil. Reduction of permeability may be relied upon to create a waterproof surface to protect underlying, water sensitive soils from the intrusion of surface water. This mechanism must be accompanied by other aspects of the geometric design into a comprehensive system. The reduction of void spaces may also tend to change the volume change under shear from a contractive to a dilative condition. The admixture type stabilizing agent also acts by binding the particles of soil together, adding cohesive shear strength and increasing the difficulty with which particles can move into a denser packing under load. Particle binding serves to reduce swelling by resisting the tendency of particles to move apart. The particles may be bound together by the action of the stabilizing agent itself (as in the case of asphalt cement), or may be cemented by chemical reaction between the soil and stabilizing agent (as in the case of lime or Portland cement). Additional improvement can arise from other chemical-physical reactions that affect the soil fabric (typically by flocculation) or the soil chemistry (typically by cation exchange). The down side of admixtures is that they require up front lab testing to confirm their performance and very good field control to obtain a uniform, long lasting product, as outlined later in this section. There are also issues of dust control and weather dependency, with some methods that should be carefully considered in the selection of these methods.

The zone that may be selected for improvement depends upon a number of factors. Among these are the depth of soft soil, anticipated traffic loads, the importance of the transportation network, constructability, and the drainage characteristics of the geometric design and the underlying soil. When only a thin zone and/or short roadway length is subject to improvement, removal and replacement will usually be the preferred alternative by most agencies, unless a suitable replacement soil is not economically available. Note that in this context, the use of the qualitative term "thin" is intentional, as the thickness of the zone can be described as thick or thin, based primarily on the project economics of the earthwork requirements and the depth of influence for the vehicle loads.

7.6.3 Thick Granular Layers

Many agencies have found that a thick granular layer is an important feature in pavement design and performance. Thick granular layers provide several benefits, including increased load-bearing capacity, frost protection, and improved drainage. While the composition of this layer takes many forms, the underlying strategy of each is to achieve desired pavement performance through improved foundation characteristics. The following sections describe the benefits of thick granular layers, typical characteristics, and considerations for the design and construction of granular embankments.

Objectives of Thick Granular Layers

Thick granular layers have been used in design for structural, drainage, and geometric reasons. Many times, a granular layer is used to provide uniformity and support as a construction platform. In areas with large quantities of readily accessible, good quality aggregates, a thick granular layer may be used as an alternative to soil stabilization. Whatever the reason, thick granular layers aim to improve the natural soil foundation. By doing this, many agencies are recognizing that the proper way to account for weak, poorly draining soils is through foundation improvement, as opposed to increasing the pavement layer thicknesses. The following is a list of objectives and benefits of thick granular layers:

- To increase the supporting capacity of weak, fine-grained subgrades.
- To provide a minimum bearing capacity for the design and construction of pavements.
- To provide uniform subgrade support over sections with highly variable soil conditions.

- To reduce the seasonal effects of moisture and temperature variations on subgrade support.
- To promote surface runoff through geometric design.
- To improve subsurface drainage and the removal of moisture from beneath the pavement layers.
- To increase the elevation of pavements in areas with high water tables.
- To provide frost protection in freezing climatic zones.
- To reduce subgrade rutting potential of flexible pavements.
- To reduce pumping and erosion beneath PCC pavements.
- To meet elevation requirements of geometric design.

Characteristics of Thick Granular Layers

Thick granular layers have been incorporated in pavement design in several ways. They can be referred to as fills or embankments, an improved or prepared subgrade, and select or preferred borrow. Occasionally, a thick granular layer is used as the pavement subbase. The two most important characteristics for all of these layers are material properties and thickness. While geometric requirements (*e.g.,* vertical profile) and improved surface runoff can be achieved by embankments constructed of any soil type, the most beneficial effects are produced through utilization of good quality, granular materials. Several methods are used to characterize the strength and stiffness of granular materials, including the California Bearing Ratio (CBR) and resilient modulus testing. In addition, several types of field plate load tests have been used to determine the composite reaction of the embankment and soil combination. In general, materials with CBR values of 20% or greater are used, corresponding to resilient moduli of approximately 120 MPa (17,500 psi). These are typically sand or granular materials, or coarse-grained materials with limited fines, corresponding to AASHTO A-1 and A-2 (GW, GP, SW and SP) soils.

Aggregate gradation and particle shape are other important properties. Typically, embankment materials are dense-graded, with a maximum top-size aggregate that varies depending on the height of the embankment. Many times, the lowest embankment layer may contain cobbles or aggregates of 100 – 200 mm (4 – 8 in.) in diameter. Granular layers placed close to the embankment surface have gradations, including maximum size aggregates, similar to subbase material specifications. Although dense-graded aggregate layers do not provide efficient drainage relative to open-graded materials, a marginal degree of subsurface seepage can be achieved by limiting the fines content to less than 10%. The type of granular material used is normally a function of material availability and cost. Pit-run gravels and crushed stone materials are the most common. The high shear strength of crushed stone is more desirable than rounded, gravelly materials; however, the use of crushed materials may not always be economically feasible.

The thicknesses of granular layers vary, depending upon their intended use. Granular layers 150 – 300 mm (6 – 12 in.) thick may be used to provide uniformity of support, or act as a construction platform for paving of asphalt and concrete layers. To increase the composite subgrade design values (*i.e.,* combination of granular layer over natural soil), it is usually necessary to place a minimum of 0.5 – 1.5 m (1½ -- 5 ft) of embankment material, depending on the strength of the granular material relative to that of the underlying soil. Likewise, granular fills placed for frost protection may also range from 0.5 – 1.5 m (1½ – 5 ft). In most cases, embankments greater than 2 m (6 ½ ft) thick have diminishing effects in terms of strength, frost protection, and drainage. Granular embankments greater than 2 – 3 m (6½ – 10 ft) thick are usually constructed for purposes of geometric design.

Considerations for Pavement Structural Design

The use of a thick granular layer presents an interesting situation for design. The placement of a granular layer of substantial thickness over a comparatively weak underlying soil forms, essentially, non-homogeneous subgrade in the vertical direction. Pavement design requires a single subgrade design value, for example CBR, resilient modulus, or k-value. This is generally determined through laboratory or field tests, when the soil mass in the zone of influence of vehicle loads is of the same type, or exhibits similar properties. In the case of a non-homogeneous subgrade, the composite reaction of the embankment and soil combination can vary from that of the natural soil to that of the granular layer. Most commonly, the composite reaction is a value somewhere between the two extremes, dependent upon the relative difference in moduli between the soil and embankment, and the thicknesses of the granular layer. The actual composite subgrade response is not known until the embankment layer is placed in the field, and it may be different once the upper pavement layers are placed.

To account for non-homogenous subgrades in pavement structural design, it is recommended to characterize the individual material properties by traditional means, such as resilient modulus or CBR testing, and to compare these results to field tests performed over the constructed embankment layers, as well as the completed pavement section. Analytical models, such as elastic layer programs, can be used to make theoretical predictions of composite subgrade response, and these predictions can then be verified by field testing. Some agencies use in-situ plate load tests to verify that a minimum composite subgrade modulus has been achieved. Deflection devices, including the Falling Weight Deflectometer (FWD), can be used for testing over the compacted embankment layer and over the constructed pavement surface.

It is advisable to use caution when selecting a design subgrade value for a non-homogenous subgrade. Experience has shown that a good-quality embankment layer must be of significant height, say 1 m (3 ft) or more, before the composite subgrade reaction begins to resemble that of the granular layer. This means that, for granular layers up to 1 m (3 ft) in height, the composite reaction can be much less than that of the embankment layer itself. If too high a subgrade design value is selected, the pavement will be under-designed. Granular layers less than 0.5 m (1.6 ft) thick have minimal impact on the composite subgrade reaction, when loaded under the completed pavement section.

7.6.4 Geotextiles and Geogrids

Geosynthetics are a class of geomaterials that are used to improve soil conditions for a number of applications. They consist of manufactured polymeric materials used in contact with soil materials or pavements as an integral part of a man-made system (after ASTM D4439). The most common applications in general use are in pavement systems for both paved and unpaved roadways, for reinforcing embankments and foundation soils, for creating barriers to water flow in liners and cutoffs, and for improving drainage. The generic term "geosynthetic" is often used to cover a wide range of different materials, including geotextiles, geogrids, and geomembranes. Combinations of these materials in layered systems are usually called geocomposites.

Geotextile and geogrid materials are the most commonly used geosynthetics in transportation, although certainly others are sometimes used. This generality is more accurate when only the pavement itself (not including the adjoining fill or cut slopes, retaining walls, abutments, or drainage facilities) is considered. Table 7-14 provides a list of transportation applications for specific basic functions of the geosynthetic. Each of these functional classes, while potentially related by the specific application being proposed, refers to an individual mechanism for the improvement of the soil subgrade. Stabilization, as reviewed in this section, is a combination of the separation, filtration, and reinforcement functions. Drainage can also play a role.

The *separation function* prevents the subgrade and the subbase from intermixing, which would most likely occur during construction and in-service due to pumping of the subgrade. The *filtration function* is required because soils requiring stabilization are usually wet and saturated. By acting as a filter, the geotextile retains the subgrade without clogging, while allowing water from the subgrade to pass up into the subbase, thus allowing destabilizing pore pressure to dissipate and promote strength gain due to consolidation. If the subbase is dirty (contains high fines), it may be desirable to use a thick, nonwoven geotextile, which will allow for *drainage* in its plane (*i.e.,* in this case, pore water pressure dissipates through the plane of the geotextile).

Geotextiles and geogrids also provide some level of *reinforcement* by laterally restraining the base or subbase and improving the bearing capacity of the system, thus decreasing shear stresses on the subgrade. Soft, weak subgrade soils provide very little lateral restraint, so when the aggregate moves or shoves laterally, ruts develop on the aggregate surface and also in the subgrade. A geogrid with good interlocking capabilities or a geotextile with good frictional capabilities can provide tensile resistance to lateral aggregate movement. The geosynthetic also increases the system bearing capacity by forcing the potential bearing surface under the wheel load to develop along alternate, longer mobilization paths and, thus, higher shear strength surfaces.

Geotextiles serve best as separators, filters and, in the case of nonwoven geotextiles, drainage layers, while geogrids are better at reinforcing. Geogrids, as with geotextiles, prevent the subbase from penetrating the subgrade, but they do not prevent the subgrade from pumping into the base. When geogrids are used, either the subbase has to be designed as a separator or a geotextile must be used in conjunction with the geogrid, either separately or as a geocomposite.

Table 7-14. Transportation uses of geosynthetic materials (after Koerner, 1998).

General Function	Typical Application
Separation of Dissimilar Materials	Between subgrade and aggregate base in paved and unpaved roads and airfields Between subgrade and ballast for railroads Between old and new asphalt layers
Reinforcement of weak materials	Over soft soils for unpaved roads, paved roads, airfield, railroads, construction platforms
Filtration	Beneath aggregate base for paved and unpaved roads and airfields or railroad ballast
Drainage	Drainage interceptor for horizontal flow Drain beneath other geosynthetic systems

Table 7-15. Appropriate subgrade conditions for stabilization using geosynthetics (after FHWA HI-95-038).

Condition	Related Measures
Poor soils	USCS of SC, CL, CH, ML, MH, OL, OH, PT or AASHTO of A-5, A-6, A-7, A-7-6
Low strength	c_u<13 psi or CBR<3 or M_R < 4500 psi
High water table	Within zone of influence of surface loads
High sensitivity	High undisturbed strength compared to remolded strength

As defined by AASHTO M288, geotextiles or geogrids in conjunction with an appropriately designed thickness of subbase aggregate provide stabilization for soft, wet subgrades with a CBR of less than 3 (a resilient modulus less than 30 MPa (4500 psi)). Table 7-15 provides subgrade conditions that are considered to be the most appropriate for geosynthetic use. These are conditions where the subgrade will not support conventional construction without substantial rutting. Engineers have compiled over 20+ years of successful use for this application in these types of conditions. Geosynthetics do not provide improvements for expansive soils, and use in stabilization for subgrade conditions that are better than those defined in Table 7-15 is questionable. However, geosynthetics may still provide a valuable function as separators for any subgrade containing large amounts of fines or as base reinforcement, even with competent subgrades, as discussed in Section 7.2.

Separation is a viable function, for soils that are seasonally weak (*e.g.*, from spring thaw) or for high fines content soils, which are susceptible to pumping. This is especially the case for permeable base applications, as covered in Section 7.2. A greater range of geotextile applicability is recognized in the M288 specification (AASHTO, 1997). With a CBR ≥ 3, the geotextile application is identified as separation. By simply maintaining the integrity of the subbase and base layers over the life of the pavement, the serviceability of the roadway section will be extended, and substantial cost benefits can be realized. Research is ongoing to quantify the cost-benefit life cycle ratio of using geosynthetics in permanent roadway systems. Initial work by Al-Qadi, 1997 indicates that the use a geosynthetic separator may increase the number of allowable design vehicles (ESALs) by a factor of two. Considering the cost of a geosynthetic is generally $1.25/m^2, while the cost of a modern pavement section is on the order of $25/m^2, the life extension of the roadway section will more than make up for the cost of the geosynthetic. In addition, as previously indicated, the geosynthetic maintains the integrity of the base such that rehabilitation should only require surface pavement restoration. **The ability of a geosynthetic to prevent premature failure and reduce long-term maintenance costs provides extremely low-cost performance insurance.**

The design of the geosynthetic for stabilization is completed using the design-by-function approach in conjunction with AASHTO M288, in the steps from FHWA HI-95-038 outlined below. A key feature of this method is the assumption that the structural pavement design is not modified at all in the procedure. The pavement design proceeds exactly according to standard procedures, as if the geosynthetic was not present. The geosynthetic instead replaces additional unbound material that might be placed to support construction operations, and replaces no part of the pavement section itself. However, this unbound layer will provide some additional support. If the soil has a CBR of less than 3, and the aggregate thickness is determined based on a low rutting criteria in the following steps, the support for the composite system is theoretically equivalent to a CBR = 3 (resilient modulus of 30 Mpa (4500 psi)). As with thick aggregate fill used for stabilization, the support value should be confirmed though field testing using, for example, a plate load test or FWD test to verify that a minimum composite subgrade modulus has been achieved. Note that the FHWA procedure is controlled by soil CBR, as measured using ASTM C4429.

1. Identify properties of the subgrade, including CBR, location of groundwater table, AASHTO and/or USCS classification, and sensitivity.
2. Compare these properties to those in Table 7-15, or with local policies. Determine if a geosynthetic will be required.
3. Design the pavement without consideration of a geosynthetic, using normal pavement structural design procedures.
4. Determine the need for additional imported aggregate to ameliorate mixing at the base/subgrade interface. If such aggregate is required, determine its thickness, t_1, and reduce the thickness by 50%, considering the use of a geosynthetic.
5. Determine additional aggregate thickness t_2 needed for establishment of a construction platform. The FHWA procedure requires the use of curves for aggregate thickness vs. the expected single tire pressure and the subgrade bearing capacity, as shown in Figure 7-21, modified for highway applications. For the purposes of this manual, the curves have been correlated with common pavement construction traffic. Select N_c based on allowable subgrade ruts, where:
   - N_c = 5 for a low rut criteria (< 50 mm (< 2 in.)),
   - N_c = 5.5 for moderate rutting (50 – 100 mm (2 – 4 in.)), and
   - N_c = 6 for large rutting (> 100 mm (> 4 in.)).

   (For comparison without a geotextile: N_c = 2.8, 3.0, or 3.3 respectively for low to large ruts.)

   Alternatively, local policies or charts may be used.
6. Select the greater of t_2 or 50% t_1.
7. Check filtration criteria for the geotextile to be used. For geogrids, check the aggregate for filtration compatibility with the subgrade (see Section 7.2), or use a

geotextile in combination with the grid meeting the following criteria. The important measures include the apparent opening size (AOS), the permeability (k), and permittivity (ψ) of the geotextile, and the 95% opening size, defined as the diameter of glass beads for which 95% will be retained on the geosynthetic. These values will be compared to a minimum standard or to the soil properties as follows

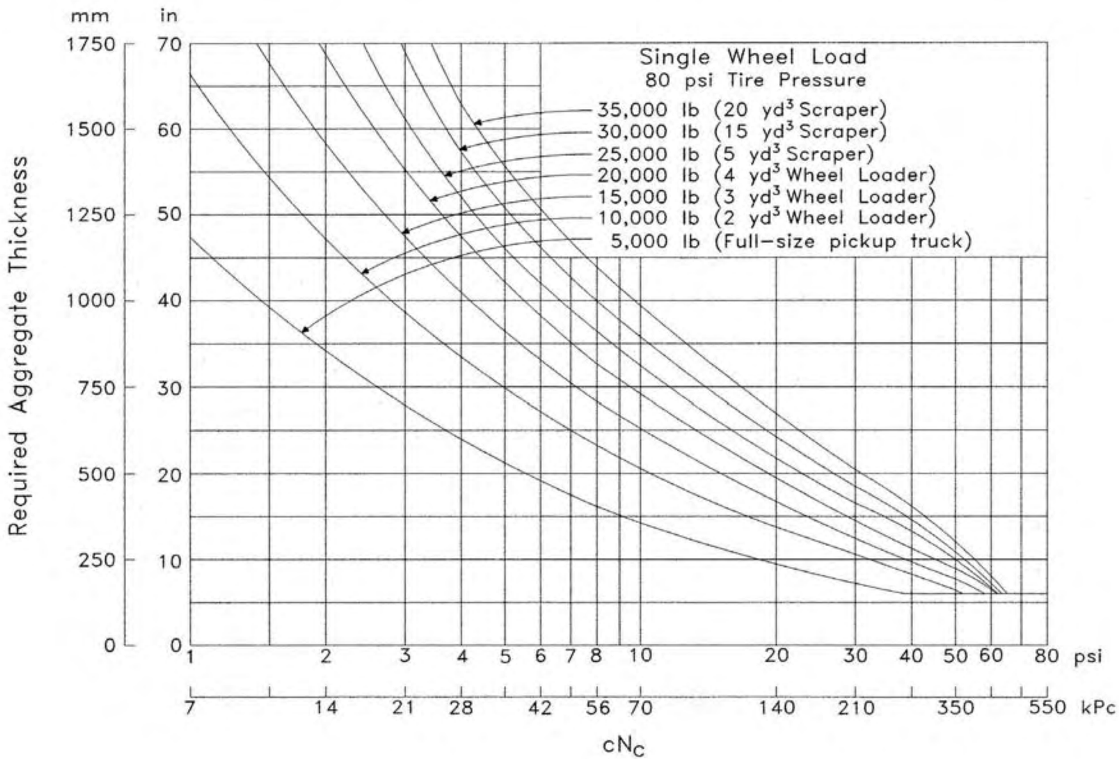
- $AOS \leq D_{85}$ (Wovens)
- $AOS \leq 1.8\ D_{85}$ (Nonwovens)
- $k_{geotextile} \geq k_{soil}$
- $\psi \geq 0.1\ sec^{-1}$

8. Determine geotextile survival criteria. The design is based on the assumption that the geosynthetic cannot function unless it survives the construction process. The AASHTO M288-99 standard categorizes the requirements for the geosynthetic based on the survival class. The requirements for the standard include the strength (grab, seam, tear, puncture, and burst), permittivity, apparent opening size, and resistance to UV degradation, based on the survival class. The survival class is determined from Table 7-5 (Section 7.2.12). For stabilization of soils, the default is Class 1, and for separation, the default is Class 2. These requirements may be reduced based on conditions and experience, as detailed in AASHTO M288. For geogrid survivability, see AASHTO PP46 and Berg et al. (2000).

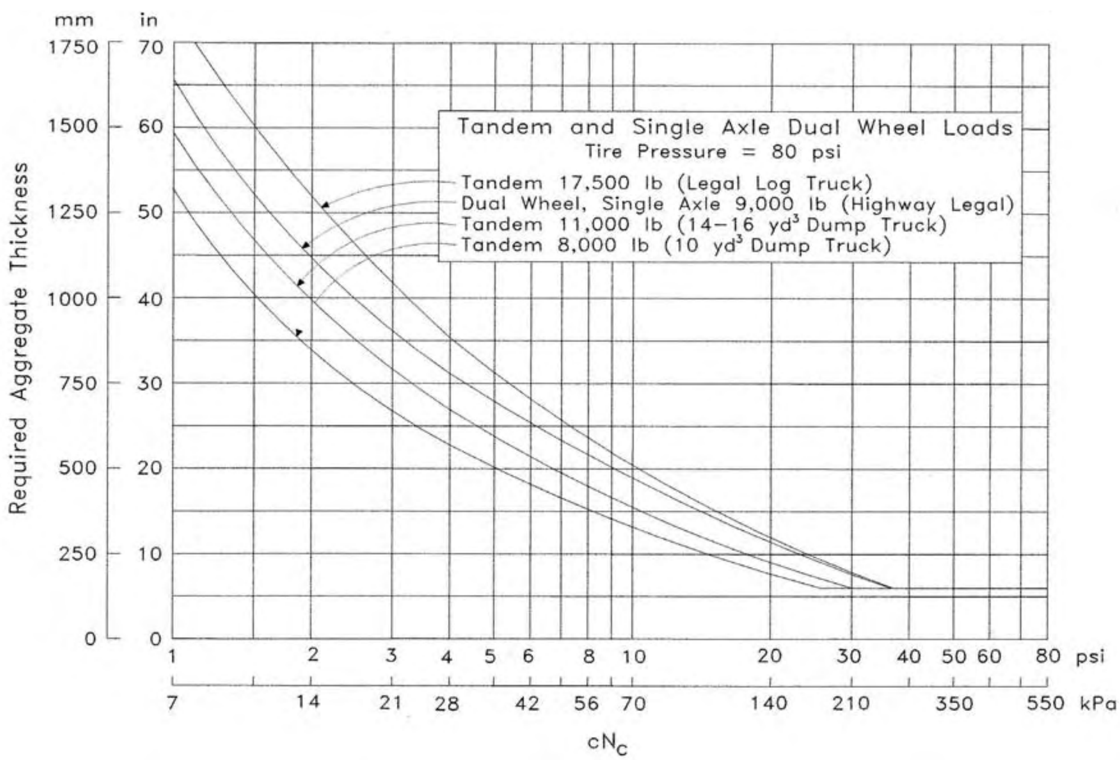
Field installation procedures introduce a number of special concerns; the AASHTO M288 standard includes a guide specification for geotextile construction. FHWA HI-905-038 (Holtz et al. 1998) recommends that this specification be modified to suit local conditions and contractors and provides example specifications. Concerns and criteria for field installation include, for example, the seam lap and sewing requirements, and construction sequencing and quality control.

7.6.5 Admixture Stabilization

As previously indicated in Section 7.6.1, there are a variety of admixtures that can be mixed with the subgrade to improve its performance. The various admixture types are shown in Table 7-15, along with initial guidance for evaluating the appropriate application of these methods. Following is a general overview of each method, followed by a generalized outline for determining the optimum admixture content requirements. Design details for each specific method are contained in Appendix F.



(a)



(b)

Figure 7-21. Thickness design curves with geosynthetics for a) single and b) dual wheel oads (after USFS, 1977, and FHWA NHI-95-038, 1998).

Table 7-16. Guide for selection of admixture stabilization method(s) (Austroads, 1998).

	MORE THAN 25% PASSING 75µm			LESS THAN 25% PASSING 75µm		
Plasticity Index	PI ≤ 10	10 < PI <20	PI ≥ 20	PI ≤ 6 PI x % passing 75µm ≤ 60	PI ≤ 10	PI > 10
Form of Stabilisation						
Cement and Cementitious Blends	Usually suitable	Doubtful	Usually not Suitable	Usually suitable	Usually suitable	Usually suitable
Lime	Doubtful	Usually suitable	Usually suitable	Usually not Suitable	Doubtful	Usually suitable
Bitumen	Doubtful	Doubtful	Usually not Suitable	Usually suitable	Usually suitable	Doubtful
Bitumen/ Cement Blends	Usually suitable	Doubtful	Usually not Suitable	Usually suitable	Usually suitable	Doubtful
Granular	Usually suitable	Usually not Suitable	Usually not Suitable	Usually suitable	Usually suitable	Doubtful
Miscellaneous Chemicals*	Usually not Suitable	Usually suitable	Usually suitable	Usually not Suitable	Doubtful	Usually suitable

Key: Usually suitable (solid black); Doubtful (vertical hatching); Usually not Suitable (white)

* Should be taken as a broad guideline only. Refer to trade literature for further information.

Note: The above forms of stabilisation may be used in combination, e.g. lime stabilisation to dry out materials and reduce their plasticity, making them suitable for other methods of stabilisation.

Lime Treatment

Lime treatment or modification consists of the application of 1 – 3% hydrated lime to aid drying of the soil and permit compaction. As such, it is useful in the construction of a "working platform" to expedite construction. Lime modification may also be considered to condition a soil for follow-on stabilization with cement or asphalt. Lime treatment of subgrade soils is intended to expedite construction, and no reduction in the required pavement thickness should be made.

Lime may also be used to treat expansive soils, as discussed in Section 7.3. Expansive soils as defined for pavement purposes are those that exhibit swell in excess of 3%. Expansion is characterized by heaving of a pavement or road when water is imbibed in the clay minerals. The plasticity characteristics of a soil often are a good indicator of the swell potential, as indicated in Table 7-17. If it has been determined that a soil has potential for excessive swell, lime treatment may be appropriate. Lime will reduce swell in an expansive soil to greater or lesser degrees, depending on the activity of the clay minerals present. The amount of lime to be added is the minimum amount that will reduce swell to acceptable limits. Procedures for conducting swell tests are indicated in the ASTM D 1883 CBR test and detailed in ASTM D 4546.

The depth to which lime should be incorporated into the soil is generally limited by the construction equipment used. However, 0.6 – 1 m (2 – 3 ft) generally is the maximum depth that can be treated directly without removal of the soil.

Lime Stabilization

Lime or pozzolonic stabilization of soils improves the strength characteristics and changes the chemical composition of some soils. The strength of fine-grained soils can be significantly improved with lime stabilization, while the strength of coarse-grained soils is usually moderately improved. Lime has been found most effective in improving workability and reducing swelling potential with highly plastic clay soils containing montmorillonite, illite, and kaolinite. Lime is also used to reduce the water content of wet soils during field compaction. In treating certain soils with lime, some soils are produced that are subject to fatigue cracking.

Lime stabilization has been found to be an effective method to reduce the volume change potential of many soils. However, lime treatment of soils can convert the soil that shows negligible to moderate frost heave into a soil that is highly susceptible to frost heave, acquiring characteristics more typically associated with silts. It has been reported that this adverse effect has been caused by an insufficient curing period. Adequate curing is also important if the strength characteristics of the soil are to be improved.

Table 7-17. Swell potential of soils (Joint Departments of the Army & Air Force, 1994).

Liquid Limit	Plasticity Index	Potential Swell
> 60	> 35	High
50 - 60	25 – 35	Marginal
< 50	< 25	Low

The most common varieties of lime for soil stabilization are hydrated lime [$Ca(OH)_2$], quicklime [CaO], and the dolomitic variations of these high-calcium limes [$Ca(OH)_2 \cdot MgO$ and $CaO \cdot MgO$]. While hydrated lime remains the most commonly used lime stabilization admixture in the U.S., use of the more caustic quicklime has grown steadily over the past two decades. Lime is usually produced by calcining[2] limestone or dolomite, although some lime—typically of more variable and poorer quality—is also produced as a byproduct of other chemical processes.

For lime stabilization of clay (or highly plastic) soils, the lime content should be from 3 – 8% of the dry weight of the soil, and the cured mass should have an unconfined compressive strength of at least 0.34 MPa (50 psi) within 28 days. The optimum lime content should be determined with the use of unconfined compressive strength and the Atterberg limits tests on laboratory lime-soil mixtures molded at varying percentages of lime. As discussed later in this section, pH can be used to determine the initial, near optimum lime content value. The pozzolanic strength gain in clay soils depends on the specific chemistry of the soil – *e.g.,* whether it can provide sufficient silica and alumina minerals to support the pozzolanic reactions. Plasticity is a rough indicator of reactivity. A plasticity index of about 10 is commonly taken as the lower limit for suitability of inorganic clays for lime stabilization. The lime-stabilized subgrade layer should be compacted to a minimum density of 95%, as defined by AASHTO T99.

Typical effects of lime stabilization on the engineering properties of a variety of natural soils are shown in Table 7-18 and Figure 7-22. These are the result of several chemical processes that occur after mixing the lime with the soil. Hydration of the lime absorbs water from the soil and causes an immediate drying effect. The addition of lime also introduces calcium (Ca^{+2}) and magnesium (Mg^{+2}) cations that exchange with the more active sodium (Na^{+}) and potassium (K^{+}) cations in the natural soil water chemistry; this cation exchange reduces the plasticity of the soil, which, in most cases, corresponds to a reduced swell and shrinkage potential, diminished susceptibility to strength loss with moisture, and improved workability. The changes in the soil-water chemistry also lead to agglomeration of particles and a coarsening of the soil gradation; plastic clay soils become more like silt or sand in texture after the addition of lime. These drying, plasticity reduction, and texture effects all occur very rapidly (usually with 1 hour after addition of lime), provided there is thorough mixing of the lime and the soil.

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[2] Calcining is the heating of limestone or dolomite to a high temperature below the melting or fusing point that decomposes the carbonates into oxides and hydroxides.

Table 7-18. Examples of the effects of lime stabilization on various soils (Rollings and Rollings, 1996).

Soil	Lime %	Atterberg Limits *LL*	Atterberg Limits *PL*	Atterberg Limits *PI*	Strength q_u[a]	Strength CBR
1. CH, residual clay[b]						
(a) Site 1, Dallas–Ft.	0	63	33	30	76	
Worth Airport,	2	62	48	14	123	
residuum from Eagle	3	60	47	13	202	
Ford shale, Britton member	4	56	46	10	323	
(b) Site 2, Dallas–	0	60	27	33	70	
Ft Worth Airport,	2	48	32	16	171	
residuum from Eagle Ford	3	45	32	13	177	
shale, Tarrant member	5	48	34	14	184	
(c) Site 3, Irving, Texas,	0	76	31	45	64	
residuum from Eagle Ford	2	61	45	16	116	
shale, Britton	3	56	45	11	193	
member	5	57	45	12	302	
2. CH, Bryce silty clay,[c]	0	53	24	29	81	
Illinois, B-horizon	3	48	27	21	201	
	5	NP	NP	NP	212	
3. CH, Appling sandy loam,[d]	0	71	33	38	92	
South Carolina, residuum	3				147	
from granite	6				171	
	8				206	
4. CH, St Ann red bauxite	0	58	25	33	119	
clay loam,[d] Jamaica,	3				127	
limestone residuum	5				334	
5. CL,[e] Pelucia Creek Dam,	0	29	18	11		
Mississippi	1	32	19	13		
	2	31	22	9		
	3	30	21	9		
6. CL, Illinoian till, Illinois,[c]	0	26	15	11	43	
glacial till	3	27	21	6	126	
	5	NP	NP	NP	126	
7. SC, sandy clay, San Lorenzo,	0	54	23	31		8
Honduras[f]	5	61	38	23		20
8. MH, Surinam red earth,[d]	0	60	32	28	72	
Surinam,	3				130	
residuum from acidic	5				136	
metamorphic rock						
9. OH, organic soil with 8.1%	0	63	27	36	4	
organics[g]	2			36	4	
	4			24	8	
	8			25	7	

[a]Unconfined compressive strength in psi at 28 days unless otherwise noted; different compaction efforts used by investigators.
[b]McCallister and Petry, 1990, accelerated curing.
[c]Thompson, 1966.
[d]Harty, 1971, 7-day cure.
[e]McElroy, 1989.
[f]Personal communication, Dr. Newel Brabston, Vicksburg, Mississippi.
[g]Arman and Munfakh, 1972, limits at 48 hours, q_u at 28 days, strength samples prepared with moisture content at the *LL*.

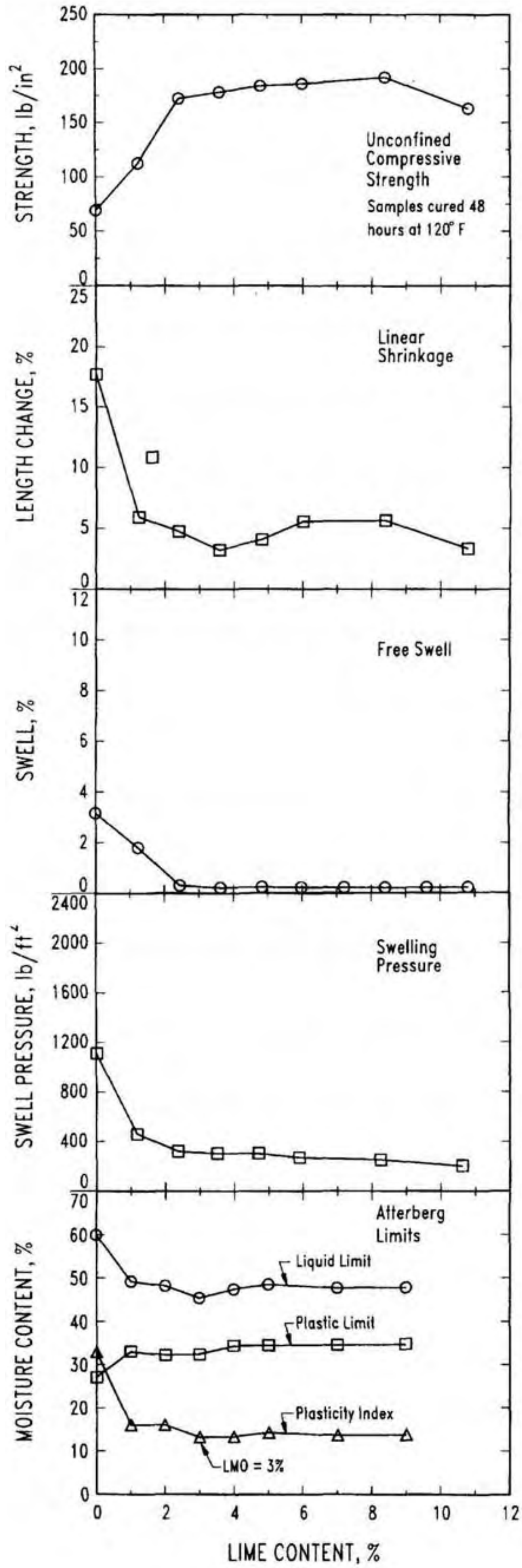


Figure 7-22. Effect of lime content on engineering properties of a CH clay (from Rollings and Rollings, 1996; from data reported by McCallister and Petry, 1990).

When soils are treated properly with lime, it has been observed that the lime-soil mixture may be subject to durability problems, the cyclic freezing and thawing of the soil. The durability of lime stabilization on swell potential and strength may be adversely affected by environmental influences:

- *Water:* Although most lime stabilized soils retain 70% to 85% of their long-term strength gains when exposed to water, there have been reported cases of poor strength retention for stabilized soils exposed to soaking. Therefore, testing of stabilized soils in the soaked condition is prudent.

- *Freeze/thaw cycles:* Freeze/thaw cycles can lead to strength deterioration, but subsequent healing often occurs where the strength loss caused by winter freeze/thaw reverses during the following warm season. The most common design approach is to specify a sufficiently high initial strength gain to retain sufficient residual strength after freeze/thaw damage.

- *Leaching:* Leaching of calcium can decrease the cation exchange in lime stabilized soil, which, in turn, can reverse the beneficial reduction in plasticity and swell potential. The potential for these effects is greater when low lime contents are used.

- *Carbonation:* If atmospheric carbon dioxide combines with lime to form calcium carbonate, the calcium silicate and calcium aluminate hydrate cements may become unstable and revert back to their original silica and alumina forms, reversing the long-term strength increase resulting from the pozzolanic reactions. Although this problem has been reported less in the United States than in other countries, its possibility should be recognized and its potential minimized by use of ample lime content, careful selection, placement, and compaction of the stabilized material to minimize carbon dioxide penetration, as well as prompt placement after lime mixing, and good curing.

- *Sulfate attack:* Sulfates present in the soil or groundwater can combine with the calcium from the lime or the alumina from the clay minerals to form ettringite, which has a volume that is more than 200% larger than that of its constituents. Massive irreversible swelling can therefore occur, and the damage it causes can be quite severe. It is difficult to predict the combinations of sulfate content, lime content, clay mineralogy and content, and environmental conditions that will trigger sulfate attack. Consequently, if there is a suspicion of possible sulfate attack, the lime stabilized soil should be tested in the laboratory to see whether it will swell when mixed and exposed to moisture.

Soils classified as CH, CL, MH, ML, SC, and GC with a plasticity index greater than 12 and with 10% passing the 0.425 mm (No. 40) sieve are potentially suitable for stabilization with lime. Lime-flyash stabilization is applicable to a broader range of soils because the cementing action of the material is less dependent on the fines contained within the soil. However, long-term durability studies of pavements with lime-flyash stabilization are rather limited.

Hydrated lime, in powder form or mixed with water as a slurry, is used most often for stabilization.

Cement Stabilization

Portland cement is widely used for stabilizing low-plasticity clays, sandy soils, and granular soils to improve the engineering properties of strength and stiffness. Increasing the cement content increases the quality of the mixture. At low cement contents, the product is generally termed cement-modified soil. A cement-modified soil has improved properties of reduced plasticity or expansive characteristics and reduced frost susceptibility. At higher cement contents, the end product is termed soil-cement or cement-treated base, subbase, or subgrade.

For soils to be stabilized with cement, proper mixing requires that the soil have a PI of less than 20% and a minimum of 45% passing the 0.425 mm (No. 40) sieve. However, highly plastic clays that have been pretreated with lime or flyash are sometimes suitable for subsequent treatment with Portland cement. For cement stabilization of granular and/or nonplastic soils, the cement content should be 3 – 10% of the dry weight of the soil, and the cured material should have an unconfined compressive strength of at least 1 MPa (150 psi) within 7 days. The Portland cement should meet the minimum requirements of AASHTO M 85. The cement-stabilized subgrade should be compacted to a minimum density of 95% as defined by AASHTO M 134.

Several different types of cement have been used successfully for stabilization of soils. Type I normal Portland cement and Type IA air-entraining cements were used extensively in the past, and produced about the same results. At the present time, Type II cement has largely replaced Type I cement as greater sulfate resistance is obtained, while the cost is often the same. High early strength cement (Type III) has been found to give a higher strength in some soils. Type III cement has a finer particle size and a different compound composition than do the other cement types. Chemical and physical property specifications for Portland cement can be found in ASTM C 150.

The presence of organic matter and/or sulfates may have a deleterious effect on soil cement. Tests are available for detection of these materials and should be conducted if their presence is suspected.

(1) *Organic matter.* A soil may be acid, neutral, or alkaline and still respond well to cement treatment. Although certain types of organic matter, such as undecomposed vegetation, may not influence stabilization adversely, organic compounds of lower molecular weight, such as nucleic acid and dextrose, act as hydration retarders and reduce strength. When such organics are present, they inhibit the normal hardening process. If the pH of a 10:1 mixture (by weight) of soil and cement 15 minutes after mixing is at least 12.0, it is probable that any organics present will not interfere with normal hardening.

(2) *Sulfates.* Although sulfate attack is known to have an adverse effect on the quality of hardened Portland cement concrete, less is known about the sulfate resistance of cement stabilized soils. The resistance to sulfate attack differs for cement-treated, coarse-grained and fine-grained soils, and is a function of sulfate concentrations. Sulfate-clay reactions can cause deterioration of fine-grained soil-cement. On the other hand, granular soil-cements do not appear susceptible to sulfate attack. In some cases, the presence of small amounts of sulfate in the soil at the time of mixing with the cement may even be beneficial. The use of sulfate-resistant cement may not improve the resistance of clay-bearing soils, but may be effective in granular soil-cements exposed to adjacent soils and/or groundwater containing high sulfate concentrations. The use of cement for fine-grained soils containing more than about 1% sulfate should be avoided.

Stabilization with Lime-Flyash (LF) and Lime-Cement-Flyash (LCF)

Stabilization of coarse-grained soils having little or no fines can often be accomplished by the use of LF or LCF combinations. Flyash, also termed coal ash, is a mineral residual from the combustion of pulverized coal. It contains silicon and aluminum compounds that, when mixed with lime and water, forms a hardened cementitious mass capable of obtaining high compressive strengths. Lime and flyash in combination can often be used successfully in stabilizing granular materials, since the flyash provides an agent with which the lime can react. Thus LF or LCF stabilization is often appropriate for base and subbase course materials.

Flyash is classified according to the type of coal from which the ash was derived. Class C flyash is derived from the burning of lignite or subbituminous coal and is often referred to as "high lime" ash because it contains a high percentage of lime. Class C flyash is self-reactive or cementitious in the presence of water, in addition to being pozzolanic. Class F flyash is derived from the burning of anthracite or bituminous coal and is sometimes referred to as "low lime" ash. It requires the addition of lime to form a pozzolanic reaction. To be acceptable quality, flyash used for stabilization must meet the requirements indicated in ASTM C 593.

Design with LF is somewhat different from stabilization with lime or cement. For a given combination of materials (aggregate, flyash, and lime), a number of factors can be varied in the mix design process, such as percentage of lime-flyash, the moisture content, and the ratio of lime to flyash. It is generally recognized that engineering characteristics such as strength and durability are directly related to the quality of the matrix material. The matrix material is that part consisting of flyash, lime, and minus No. 4 aggregate fines. Basically, higher strength and improved durability are achievable when the matrix material is able to "float" the coarse aggregate particles. In effect, the fine size particles overfill the void spaces between the coarse aggregate particles. For each coarse aggregate material, there is a quantity of matrix required to effectively fill the available void spaces and to "float" the coarse aggregate particles. The quantity of matrix required for maximum dry density of the total mixture is referred to as the optimum fines content. In LF mixtures, it is recommended that the quantity of matrix be approximately 2% above the optimum fines content. At the recommended fines content, the strength development is also influenced by the ratio of lime to flyash. Adjustment of the lime-flyash ratio will yield different values of strength and durability properties.

Asphalt Stabilization

Generally, asphalt-stabilized soils are used for base and subbase construction. Use of asphalt as a stabilizing agent produces different effects, depending on the soil, and may be divided into three major groups: 1) sand-bitumen, which produces strength in cohesionless soils, such as clean sands, or acts as a binder or cementing agent, 2) soil-bitumen, which stabilizes the moisture content of cohesive fine-grained soils, and 3) sand-gravel bitumen, which provides cohesive strength and waterproofs pit-run gravelly soils with inherent frictional strength. The durability of bitumen-stabilized mixtures generally can be assessed by measurement of their water absorption characteristics. Treatment of soils containing fines in excess of 20% is not recommended.

Stabilization of soils and aggregates with asphalt differs greatly from cement and lime stabilization. The basic mechanism involved in asphalt stabilization of fine-grained soils is a waterproofing phenomenon. Soil particles or soil agglomerates are coated with asphalt that prevents or slows the penetration of water that could normally result in a decrease in soil strength. In addition, asphalt stabilization can improve durability characteristics by making the soil resistant to the detrimental effects of water, such as volume. In noncohesive materials, such as sands and gravel, crushed gravel, and crushed stone, two basic mechanisms are active: waterproofing and adhesion. The asphalt coating on the cohesionless materials provides a membrane that prevents or hinders the penetration of water and thereby reduces the tendency of the material to lose strength in the presence of water. The second mechanism has been identified as adhesion. The aggregate particles adhere to the asphalt and the asphalt acts as a binder or cement. The cementing effect thus increases shear strength by increasing cohesion. Criteria for design of bituminous-stabilized soils and aggregates are based almost entirely on stability and gradation requirements. Freeze-thaw and wet-dry durability tests are not applicable for asphalt-stabilized mixtures.

There are three basic types of bituminous-stabilized soils, including

(1) *Sand bitumen.* A mixture of sand and bitumen in which the sand particles are cemented together to provide a material of increased stability.
(2) *Gravel or crushed aggregate bitumen.* A mixture of bitumen and a well-graded gravel or crushed aggregate that, after compaction, provides a highly stable waterproof mass of subbase or base course quality.
(3) *Bitumen lime.* A mixture of soil, lime, and bitumen that, after compaction, may exhibit the characteristics of any of the bitumen-treated materials indicated above. Lime is used with materials that have a high PI, *i.e.,* above 10.

Bituminous stabilization is generally accomplished using asphalt cement, cutback asphalt, or asphalt emulsions. The type of bitumen to be used depends upon the type of soil to be stabilized, method of construction, and weather conditions. In frost areas, the use of tar as a binder should be avoided because of its high temperature susceptibility. Asphalts are affected to a lesser extent by temperature changes, but a grade of asphalt suitable to the prevailing climate should be selected. As a general rule, the most satisfactory results are obtained when the most viscous liquid asphalt that can be readily mixed into the soil is used. For higher quality mixes in which a central plant is used, viscosity-grade asphalt cements should be used. Much bituminous stabilization is performed in-place, with the bitumen being applied directly on the soil or soil aggregate system, and the mixing and compaction operations being conducted immediately thereafter. For this type of construction, liquid asphalts, *i.e.,* cutbacks and emulsions, are used. Emulsions are preferred over cutbacks because of energy constraints and pollution control efforts. The specific type and grade of bitumen will depend on the characteristics of the aggregate, the type of construction equipment, and the climatic conditions. Generally, the following types of bituminous materials will be used for the soil gradation indicated:

(1) Open-graded aggregate.
    a. Rapid- and medium-curing liquid asphalts RC-250, RC-800, and MC-3000.
    b. Medium-setting asphalt emulsion MS-2 and CMS-2.

(2) Well-graded aggregate with little or no material passing the 0.075 mm (No. 200) sieve.
    a. Rapid and medium-curing liquid asphalts RC-250, RC-800, MC-250, and MC-800.
    b. Slow-curing liquid asphalts SC-250 and SC-800.
    c. Medium-setting and slow-setting asphalt emulsions MS-2, CMS-2, SS-1, and CSS-1.

(3) Aggregate with a considerable percentage of fine aggregates and material passing the 0.075 mm (No. 200) sieve.
    a. Medium-curing liquid asphalt MC-250 and MC-800.
    b. Slow-curing liquid asphalts SC-250 and SC-800
    c. Slow-setting asphalt emulsions SS-1, SS-01h, CSS-1, and CSS-lh.

The simplest type of bituminous stabilization is the application of liquid asphalt to the surface of an unbound aggregate road. For this type of operation, the slow- and medium-curing liquid asphalts SC-70, SC-250, MC-70, and MC-250 are used.

The recommended soil gradations for subgrade materials and base or subbase course materials are shown in Tables 7-19 and 7-20, respectively.

Table 7-19. Recommended gradations for bituminous-stabilized subgrade materials (Joint Departments of the Army and Air Force, 1994).

Sieve Size	Percent Passing
75-mm (3-in.)	100
4.75-mm (#4)	50-100
600-µm (#30)	38-100
75-µm (#200)	2-30

Table 7-20. Recommended gradations for bituminous-stabilized base and subbase materials (Joint Departments of the Army and Air Force, 1994).

Sieve Size	37.5 mm (1 ½ in.) Maximum	25 mm (1-in.) Maximum	19 mm (¾-in.) Maximum	12.7 mm (½-in.) Maximum
37.5-mm (1½-in.)	100	-	-	-
25-mm (l-in.)	8 4 ± 9	100	-	-
19-mm (¾-in.)	76 ± 9	83 ± 9	100	-
M-in	66 ± 9	73 ± 9	82 ± 9	100
9.5-mm (3/8-in.)	59 ± 9	64 ± 9	72 ± 9	83 ± 9
0.475-mm (#4)	45 ± 9	48 ± 9	54 ± 9	62 ± 9
2.36-mm (#8)	35 ± 9	36 ± 9	41 ± 9	47 ± 9
1.18-mm (#16)	27 ± 9	28 ± 9	32 ± 9	36 ± 9
600-µm (#30)	20 ± 9	21 ± 9	24 ± 9	28 ± 9
300-µm (#50)	14 ± 7	16 ± 7	17 ± 7	20 ± 7
150-µm (#100)	9 ± 5	11 ± 5	12 ± 5	14 ± 5
75-µm (#200)	5 ± 2	5 ± 2	5 ± 2	5 ± 2

Stabilization with Lime-Cement and Lime-Bitumen

The advantage of using combination stabilizers is that one of the stabilizers in the combination compensates for the lack of effectiveness of the other in treating a particular aspect or characteristic of a given soil. For instance, in clay areas devoid of base material, lime has been used jointly with other stabilizers, notably Portland cement or asphalt, to provide acceptable base courses. Since Portland cement or asphalt cannot be mixed successfully with plastic clays, the lime is added first to reduce the plasticity of the clay. While such stabilization practice might be more costly than the conventional single stabilizer methods, it may still prove to be economical in areas where base aggregate costs are high. Two combination stabilizers are considered in this section: lime-cement and lime-asphalt.

a) *Lime-cement.* Lime can be used as an initial additive with Portland cement, or as the primary stabilizer. The main purpose of lime is to improve workability characteristics, mainly by reducing the plasticity of the soil. The design approach is to add enough lime to improve workability and to reduce the plasticity index to acceptable levels. The design lime content is the minimum that achieves desired results. The design cement content is determined following procedures for cement-stabilized soils presented in Appendix F.

b) *Lime-asphalt.* Lime can be used as an initial additive with asphalt, or as the primary stabilizer. The main purpose of lime is to improve workability characteristics and to act as an anti-stripping agent. In the latter capacity, the lime acts to neutralize acidic chemicals in the soil or aggregate that tend to interfere with bonding of the asphalt. Generally, about 1 – 2% percent lime is all that is needed for this objective. Since asphalt is the primary stabilizer, the procedures for asphalt-stabilized materials, as presented Appendix F, should be followed.

Admixture Design

Design of admixtures takes on a similar process regardless of the admixture type. The following steps are generally followed and are generic to lime, cement, L-FA and L-C-FA, or asphalt admixtures.

Step 1. Classify soil to be stabilized.
(% < 0.075 mm – No. 200 sieve, % < 0.425 mm – No. 40 Sieve, PI, etc.)

Step 2. Prepare trial mixes with varying % content.
Lime: Select lowest % with pH = 12.4 in 1 hour
Cement: Use table to estimate cement content requirements
Asphalt: Use equation & table in Appendix F to estimate the quantity of cutback asphalt

Step 3. Develop moisture-density relationship for initial design.

Step 4. Prepare triplicate samples and cure specimens at target density.
Use optimum water content and % initial admixture, +2% and +4%

Step 5. Determine index strength.
Lime and Cement: Determine unconfined compressive strength (ASTM D 5102)
Asphalt: Determine Marshall stability

Step 6. Determine resilient modulus for optimum percent admixture.
Perform test or estimate using correlations (See Chapter 5)

Step 7. Conduct freeze-thaw tests (Regional as required).
(For Cement, CFA, L-C-FA)

Step 8. Select % to achieve minimum design strength and F-T durability.

Step 9. Add 0.5 – 1% to compensate for non-uniform mixing.

Appendix F provides specific design requirements and design step details for each type of admixture reviewed in this section. Additional design and construction information can also be obtained from industry publications including

- *Soil-Cement Construction Handbook*, Portland Cement Association, Skokie, Il, 1995.
- *Lime-Treated Soil Construction Manual: Lime Stabilization & Lime Modification,* National Lime Associaiton, Arlington, Virginia, 2004.
- *Flexible Pavement Manual*, American Coal Ash Association, Washington, D.C., 1991.
- *A Basic Emulsion Manual*, Asphalt Institute, Manual Series #19.
- http://www.cement.org/index.asp
- http://www.lime.org/

7.6.6 Soil Encapsulation

Soil encapsulation is a foundation improvement technique that has been used to protect moisture sensitive soils from large variations in moisture content. The concept of soil encapsulation is to keep the fine-grained soils at or slightly below optimum moisture content, where the strength of these soils can support heavier trucks and traffic. This technique has been used by a number of states (*e.g.,* Texas and Wyoming) on selected projects to improve the foundations of higher volume roadways. It is more commonly used as a technique in Europe and in foundation or subbase layers for low-volume roadways, where the import of higher quality paving materials is restricted from a cost standpoint. More than 100 projects have been identified around the world, usually reporting success in controlling expansive soils (Steinberg, 1998).

Fine-grained soils can provide adequate bearing strengths for use as structural layers in pavements and embankments, as long as the moisture content remains below the optimum moisture content. However, increases in moisture content above the optimum value can cause a significant reduction in the stiffness (*i.e.,* resilient modulus) and strength of fine-grained materials and soils. Increased moisture content in fine-grained soils below pavements occurs over time, especially in areas subject to frost penetration and freeze-thaw cycles. Thus, fine-grained soils cannot be used as a base or subbase layer unless the soils are protected from any increase in moisture.

The soil encapsulation concept, sometimes referred to as membrane encapsulated soil layer (MESL), is a method for maintaining the moisture content of the soil at the desired level by encapsulating the soil in waterproof membranes. The waterproof membranes prevent water from infiltrating the moisture sensitive material. The resilient modulus measured at or below optimum conditions remains relatively constant over the design life of the pavement.

The prepared subgrade is normally sprayed with an asphalt emulsion before the bottom membrane of polyethylene is placed. This asphalt emulsion provides added waterproofing protection in the event the membrane is punctured during construction operations, and acts as an adhesive for the membrane to be placed in windy conditions. The first layer of soil is placed in sufficient thickness such that the construction equipment will not displace the underlying material. The completed soil embankment is also sprayed with an asphalt emulsion before placement of the top membrane. To form a complete encapsulation, the bottom membrane is brought up the sides and wrapped around the top, for an excavated section, or the top membrane is draped over the sides, for an embankment situation. The top of the membrane is sprayed with the same asphalt emulsion and covered with a thin layer of clean sand to blot the asphalt and to provide added protection against puncture by the construction equipment used to place the upper paving layers.

The reliability of this method to maintain the resilient modulus and strength of the foundation soil over long periods of time is unknown. More importantly, roadway maintenance and the installation of utilities in areas over time limit the use of this technique. Thus, this improvement technique is not suggested unless there is no other option available.
If this technique is used, the pavement designer should be cautioned regarding the use of the environmental effects model (EICM) to predict changes in moisture over time. Special design computations will be needed to restrict the change in moisture content of the MESL over time. The resilient modulus used in design for the MESL should be held constant over the design life of the pavement. The designer should also remember that any utilities placed after pavement construction could make that assumption invalid.

7.6.7 Lightweight Fill

When constructing pavements on soft soils, there is always a concern for settlement. For deeper deposits where shallow surface stabilization may not be effective, thicker granular aggregate as discussed in Section 7.3, may be effective for control deformation under wheel load, but would increase the concern for settlement. An alternate to replacement with aggregate would be to use lightweight fill.

The compacted unit density of most soil deposits consisting of sands, silts, or clays ranges from about 1,800 – 2,200 kg/m^3 (112 – 137 lbs/ft^3) Lightweight fill materials are available from the lower end of this range down to 12 kg/m^3 (0.75 lbs/ft^3). In many cases, the use of lighter weight materials on soft soils will likely result in both reduced settlement and increased stability. The worldwide interest and use of lightweight fill materials has led to the recent publication by the Permanent International Association of Road Congresses (PIARC) of an authoritative reference *"Lightweight Filling Materials"* in 1997.

Many types of lightweight fill materials have been used for roadway construction. Some of the more common lightweight fills are listed in Table 7-21. There is a wide range in density of the lightweight fill materials, but all have a density less than conventional soils. Addtional information on the composition and sources of the lightweight fill materials listed in Table 7-21 can be found in FHWA NHI-04-001 Ground Improvement Methods technical summaries.

Some lightweight fill materials have been used for decades, while others are relatively recent developments. Wood fiber has been used for many years by timber companies for roadways crossing peat bogs and low-lying land, as well as for repair of slide zones.

The steel-making companies have produced slag since the start of the iron and steel making industry. Initially, the slag were stockpiled as waste materials, but beginning around 1950, the slag were crushed, graded, and sold for fill materials.

Geofoam is a generic term used to describe any foam material used in a geotechnical application. Geofoam includes expanded polystyrene (EPS), extruded polystyrene (XPS), and glassfoam (cellular glass). Geofoam was initially developed for insulation material to prevent frost from penetrating soils. The initial use for this purpose was in Scandinavia and North America in the early 1960s. In 1972, the use of geofoam was extended as a lightweight fill for a project in Norway.

The technique of using pumping equipment to inject foaming agents into concrete was developed in the late 1930s. Little is known about the early uses of this product. However, the U.S. Army Corps of Engineers used foamed concrete as a tunnel lining and annular fill. This product is generally job-produced as a cement/water slurry with preformed foam blended for accurate control and immediate placement.

Table 7-21. Densities and approximate costs for various lightweight fill materials.

Fill Type	Range in Density kg/m^3	Range in Specific Gravity	Approximate Cost[1] \$/m^3
Geofoam (EPS)	12 to 32	0.01 to .03	40.00 to 85.00[2]
Foamed Concrete	320 to 970	0.3 to 0.8	40.00 to 55.00
Wood Fiber	550 to 960	0.6 to 1.0	12.00 to 20.00[2]
Shredded Tires	600 to 900	0.6 to 0.9	20.00 to 30.00[2]
Expanded Shale And Clay	600 to 1040	0.6 to 1.0	40.00 to 55.00[3]
Flyash	1120 to 1440	1.1 to 1.4	15.00 to 21.00[3]
Boiler Slag	1000 to 1750	1.0 to 1.8	3.00 to 4.00[3]
Air-Cooled Slag	1100 to 1500	1.1 to 1.5	7.50 to 9.00[3]

[1] See Chapter 6 for details on cost data
[2] Price includes transportation and placement cost
[3] FOB plant

Shredded tires and tire bales are a relatively recent source of lightweight fill materials. The availability of this material is increasing each year, and its use as a lightweight fill is further promoted by the need to dispose of tires. In most locations, the tires are stockpiled, but they are unsightly and present a serious fire and health hazard. Shredded tires have been used for lightweight fill in the United States and in other countries since the mid 1980s. More than 85 fills using shredded tires as a lightweight fill have been constructed in the United States. In 1995, three tire shred fills with a thickness greater than 8 m (26 ft) experienced an unexpected internal heating reaction. As a result, FHWA issued an Interim Guideline to minimize internal heating of tire shred fills in 1997, limiting tire shred layers to 3 m (9.8 ft).

Expanded shale lightweight aggregate has been used for decades to produce aggregate for concrete and masonry units. Beginning in about 1980, lightweight aggregates have also been used for geotechnical purposes. Completed projects include the Port of Albany, New York marine terminal, where lightweight fill was used behind a bulkhead to reduce the lateral pressures on the steel sheeting. Other projects include construction of roadways over soft ground. The existing high-density soils were partially removed and replaced with lightweight aggregate to reduce settlement. Other projects have included improvement of slope stability by reduction of the gravitational driving force of the soil in the slope and replacement with a lightweight fill.

Waste products from coal burning include flyash and boiler slag. Both of these materials have been used in roadway construction. One of the first documented uses of flyash in an engineered highway embankment occurred in England in 1950. Trial embankments led to the acceptance of flyash fills, and other roadway projects were constructed in other European countries. In 1965, a flyash roadway embankment was constructed in Illinois. In 1984, a project survey found that flyash was used in the construction of 33 embankments and 31 area fills. Boiler slag has been used for backfill since the early 1970s. Many state highway department specifications allow the use of boiler slag as an acceptable fine or coarse aggregate.

The FHWA NHI-04-001 provides an overview of the more common lightweight fill materials that have been used for geotechnical applications in highway construction. Typical geotechnical engineering parameters that are important for design are provided. In addition, design and construction considerations unique to each of these lightweight fill materials are presented. This information can be used for preliminary planning purposes. The technical summary also presents guidelines for preparation of specifications along with suggested construction control procedures. Four case histories are also presented to demonstrate the effectiveness of lightweight fills for specific situations. Approximate costs for the various lightweight fill materials are also presented.

With regard to pavement design, if a minimum of 1 m (3 ft) of good quality gravel type fill is placed between the pavement structure and the lightweight materials as a cover, then the lightweight material will have little impact on pavement design, even for the more compressible tire and geofoam materials. However, if a thinner cover must be used, the support value for these materials must be determined. Lab tests can be used, as discussed in Chapter 5, especially for the granular type materials. The ideal method is to perform field resilient modulus tests on placed material (*i.e.,* on cover soils after placement over the lightweight material(s)), especially for the bulkier materials, such as tires and geofoam.

7.6.8 Deep Foundations and Other Foundation Improvement Methods (from Elias et al., 2004)

In some cases, the extent (area and depth) of poor subgrade conditions are too large for surface stabilization or removal. In extreme cases, the soils may be too week to support the roadway embankment (even for embankments that only consist of the pavement structure). In these cases, other deep ground improvement methods, such as deep foundations, may be required. Ground improvement technologies are geotechnical construction methods used to alter and improve poor ground conditions so that embankment and structure construction can meet project performance requirements where soil replacement is not feasible for environmental or technical reasons, or it is too costly.

Ground improvement has one or more than one of the following main functions:

- to increase bearing capacity, shear or frictional strength,
- to increase density,
- to control deformations,
- to accelerate consolidation,
- to decrease imposed loads,
- to provide lateral stability,
- to form seepage cutoffs or fill voids,
- to increase resistance to liquefaction and,
- to transfer embankment loads to more competent layers

There are three strategies available to accomplish the above functions representing different approaches. The first method is to increase the shear strength, density, and/or decrease the compressibility of the foundation soil. The second method is to utilize a lightweight fill embankment to reduce significantly the applied load to the foundation, and the third method is to transfer loads to a more competent deeper layer.

The selection of candidate ground improvement methods for any specific project follows a sequential process. The steps in the process include a sequence of evaluations that proceed from simple to more detailed, allowing a best method to emerge. The process is described as follows:

1) *Identify potential poor ground conditions, their extent, and type of negative impact*. Poor ground conditions are typically characterized by soft or loose foundation soils, which, under load, would cause long-term settlement, or cause construction or post-construction instability.
2) *Identify or establish performance requirements.* Performance requirements generally consist of deformation limits (horizontal and vertical), as well as some minimum factors of safety for stability. The available time for construction is also a performance requirement.
3) *Identify and assess any space or environmental constraints.* Space constraints typically refer to accessibility for construction equipment to operate safely, and environmental constraints may include the disposal of spoil (hazardous or not hazardous) and the effect of construction vibrations or noise.
4) *Assessment of subsurface conditions*. The type, depth, and extent of the poor soils must be considered, as well as the location of the ground-water table. It is further valuable to have at least a preliminary assessment of the shear strength and compressibility of the identified poor soils.
5) *Preliminary selection*. Preliminary selection of potentially applicable method(s) is generally made on a qualitative basis, taking into consideration the performance

criteria, limitations imposed by subsurface conditions, schedule and environmental constraints, and the level of improvement that is required. Table 7-22, which groups the available methods in six broad categories, can be used as a guide in this process to identify possible methods and eliminate those that by themselves, or in conjunction with other methods, cannot produce the desired performance.

6) *Preliminary design*. A preliminary design is developed for each method identified under “Preliminary selection” and a cost estimate prepared on the basis of data in Table 7-23. The guidance in developing preliminary designs is contained within each Technical Summary.
7) *Comparison and selection*. The selected methods are then compared, and a selection made by considering performance, constructability, cost, and other relevant project factors.

State-of-the-art design and construction methods and/or references are provided in each of the FHWA NHI-04-001 Ground Improvement Methods technical summaries to form the basis of a final design. The success of any ground improvement method is predicated on the implementation of a QA/QC program to verify that the desired foundation improvement level has been reached. These programs incorporate a combination of construction observations, in-situ testing and laboratory testing to evaluate the treated soil in the field. Details are provided in each technical summary contained in the FHWA NHI-04-001.

Table 7-22. Ground improvement categories, functions, methods and applications (Elias et al., 2004).

Category	Function	Methods	Comment
Consolidation	Accelerate consolidation, increase shear strength	(1) Wick drains (2) Vacuum consolidation	Viable for normally consolidated clays. Vacuum consolidation viable for very soft clays. Can achieve up to 90% consolidation in a few months.
Load Reduction	Reduce load on foundation, reduce settlement	(1) Geofoam, (2) Foamed concrete (3) Lightweight granular fills, tire chips, etc.	Density varies from 1 – 12 kN/m^3 (6 – 76 lb/ft^3). Granular fills usage subject to local availability.
Densification	Increase density, bearing capacity and frictional strength of granular soils. Decrease settlement and increase resistance to liquefaction.	(1) Vibro-compaction using vibrators (2) Dynamic compaction by falling weight impact	Vibrocompaction viable for clean sands with <15% fines. Dynamic compaction limited to depths of about 10 m (33 ft), but is applicable for a wider range of soils. Both methods can densify granular soils up to 80% Relative Density. Dynamic compaction generates vibrations for a considerable lateral distance.
Reinforcement	Internally reinforces fills and/or cuts. In soft foundation soils, increases shear strength, resistance to liquefaction and decreases compressibility.	(1) MSE retaining walls (2) Soil Nailing walls (3) Stone column to reinforce foundations	Soil Nailing may not applicable in soft clays or loose fills. Stone columns applicable in soft clay profiles to increase global shear strength and reduce settlement.
Chemical Stabilization by Deep Mixing Methods	Physio-chemical alteration of foundation soils to increase their tensile, compressive and shear strength, and to decrease settlement and/or provide lateral stability and or confinement.	(1) Wet mixing methods using primarily cement (2) Dry mixing methods using lime-cement	Applicable in soft to medium stiff clays for excavation support where the groundwater table must be maintained, or for foundation support where lateral restraint must be provided, or to increase global stability and decrease settlement. Requires significant QA/QC program for verification.

Table 7-22. Ground improvement categories, functions, methods and applications (continued).

Category	Function	Methods	Comment
Chemical Stabilization by Grouting	To form seepage cutoffs, fill voids, increase density, increase tensile and compressive strength	(1) Permeation grouting with particulate or chemical grouts (2) Compaction grouting (3) Jet grouting, and (4) Bulk filling	(1) Permeation grouting to increase shear strength or for seepage control, (2) compaction grouting for densification and (3) jet grouting to increase tensile and/or compressive strength of foundations, and (4) bulk filling of any subsurface voids.
Load Transfer	Transfer load to deeper bearing layer	Column (Pile) supported embankments on flexible geosynthetic mats	Applicable for deep soft soil profiles or where a tight schedule must be maintained. A variety of stiff or semi-stiff piles can be used.

Table 7-23a. Comparative Costs (SI units) (Elias et al., 2004).

Method	Unit Cost	Cost of Treated Volume $/m^3
Wick Drains	\$ 1.50 - 4.00/m	\$ 0.80 - 1.60
Lightweight Fill Granular Tires-Wood Geofoam Foamed Concrete	 \$ 3.00 - 21.00/m^3 \$ 12.00 - 30.00/m^3 \$ 35.00 - 65.00/m^3 \$ 45.00 - 65.00/m^3	
Vibrocompaction	\$ 15.00 - 25.00/m	\$ 1.00 - 4.00
Dynamic Compaction	\$ 6.00 - 11.00/m^2	\$ 1.00 - 2.00
MSE Walls	\$ 160.00 - 300.00/m^2	
RSS Slopes	\$ 110.00 - 260.00/m^2	
Soil Nail Walls	\$ 400.00 - 600.00/m^2	
Stone Columns	\$ 40.00 - 60.00/m	\$ 50 - 75
Deep Soil Mixing Dry w/lime-cement Wet w/cement	 \$30.00/m 	 \$ 60 \$ 85 -150
Grouting Permeation Compaction Jet	 \$ 65.00/m + \$ 0.70/Liter 	 \$ 30 - 200 \$ 200 - 275
Column-Supported Embankments	\$ 95/m^2 + cost of column	n/a

Table 7-23b. Comparative Costs (U.S. customary units) (Elias et al., 2004).

Method	Unit Cost	Cost of Treated Volume $/yd^3
Wick Drains	\$ 0.46 - 1.22/ft	\$ 0.60 - 1.20
Lightweight Fill Granular Tires-Wood Geofoam Foamed Concrete	 \$ 2.30 - 16.10/yd^3 \$ 9.20 - 23.00/yd^3 \$ 26.75 - 50.00/yd^3 \$ 34.50 - 50.00/yd^3	
Vibrocompaction	\$ 4.60 - 7.60/ft	\$ 0.75 – 3.00
Dynamic Compaction	\$ 5.00 - 9.20/yd^2	\$ 0.75 - 1.50
MSE Walls	\$ 15.00 - 28.00/ft^2	
RSS Slopes	\$ 10.00 - 24.00/ft^2	
Soil Nail Walls	\$ 37.00 - 56.00/ft^2	
Stone Columns	\$ 12.20 - 18.30/ft	\$ 38 - 57
Deep Soil Mixing Dry w/lime-cement Wet w/cement	 \$9.15/ft 	 \$ 46 \$ 65 -115
Grouting Permeation Compaction Jet	 \$ 20/ft + \$ 2.65/Gallon 	 \$ 23 - 153 \$ 150 - 210
Column Supported Embankments	\$ 81.50/yd^2 + cost of column	n/a

7.7 RECYCLE

Recycling, in principal, is a very powerful and often political concept. While the benefits of recycling including conservation of aggregate and binders and preservation of the environment, it requires serious consideration. The long-term performance of recycled materials in pavements and, in come cases the environmental impact, must be carefully evaluated to avoid costly performance and maintenance issues. In this section, the evaluation requirements for recycled materials will be reviewed. There are two forms of recycling in pavements: 1) reuse of the pavement materials themselves and 2) the use of recycled waste materials for subgrade stabilization or as a substitute for aggregate.

7.7.1 Pavement Recycling

The method of recycling the pavement will, in most cases, depend on whether the surface pavement has an AC or PCC surface pavement. In either case, the material could be rubblized, or, in some cases, processed (*e.g.,* sieving, stockpiling, and reusing the reclaimed asphalt pavement (RCP) materials or recycled concrete materials (RCM) plus the aggregate base). Both pavement types can also be rubblized in place and compacted. This procedure is known as rubblize and roll for PCC pavements and full-depth reclamation for AC pavements. For AC pavement materials, there are also several other methods, including hot mix asphalt recycling, hot in-place recycling, and cold in-place recycling, all of which produce a bound product, which is beyond the scope of this manual.

Recycled Asphalt

The design requirements for RCP aggregates are essentially the same as natural aggregates. The strength of the material must be determined using the methods outlined in Chapter 5 and Section 7.3, and an assessment must be made of the drainage characteristics, as discussed in Section 7.2. With full-depth reclamation, all of the asphalt pavement sections and a predetemined amount of underlying materials are treated with recyling agents to produce a stabilized base course, and is well covered in FHWA-SA-98-042 (Kandhal, and Mallick, 1997) . The advantages of this process are establishing high production rate and maintaining the geometry of the pavement or shoulder reconstruction. The primary drawbacks are aggregate size, depth limitation and depth control, and need for specialized equipment. With the sizing, RAP can often only be effectively screened down to a maximum size of 50 mm (2 in.). If a significant amount of contaminated base course (*i.e.,* containing significant amount of fines) is removed with the asphalt, the hydraulic properties of the aggregate could also be poor.

Recycled Concrete

Again, the design requirements for RCM aggregates are essentially the same as natural aggregates. Recycled concrete has been used by a number of states as base materials since the 1980s. However, several states have identified three significant issues, including

- the formation of tufa (calcium deposits) clogging drains and filter materials;
- alkaline (high pH) run-off; and,
- freeze thaw degradation.

As a result, these states are now primarily using the recycled concrete, mixed with natural soils, as embankment fill.

7.7.2 Recycled Waste Materials

A number of recylced waste materials have been used in permanent construction, practically all of which where covered in Section 7.6.7 since they have a lighter weight than conventional aggregate. Other applications not reviewed in Section 7.6.7 include the use of recycled materials as a replacement for base materials (*e.g.,* slag and bottom ash) and, in some cases (*e.g.,* glass and tire shreds) drainage aggregate. As indicated in Section 7.6.7, the materials must be evaluated with respect to the same property requirements as the material they will replace. The pavement support value (*e.g.,* resilient modulus or CBR) should be determine based on lab tests reviewed in Chapter 5. Field trails using FWD tests to confirm the as constructed properties are also recommended. Durability is a critical issue with many of these materials, and, obviously, an assessment of environmental issues must be made.

7.8 REFERENCES

AASHTO (1993). *AASHTO Guide for Design of Pavement Structures*, American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO (2004). *Standard Specifications for Transportation Materials and Methods of Sampling and Testing (24rd ed.)*, American Association of State Highway and Transportation Officials, Washington, D.C.

American Society for Testing & Materials (2000). *ASTM Book of Standards*, *Vol. 4, Section 01*, Construction Materials: Cement; Lime; Gypsum, ASTM, Philadelphia, PA.

American Society for Testing & Materials (2000). *ASTM Book of Standards*, *Vol. 4, Section 02*, Construction Materials: Concrete and Aggregates, ASTM, Philadelphia, PA.

American Society for Testing & Materials (2000). *ASTM Book of Standards*, *Vol. 4, Section 08 and 09*, Construction Materials: Soils & Rocks, ASTM, Philadelphia, PA.

American Society for Testing & Materials (2000). *ASTM Book of Standards*, *Vol. 4, Section 03*, Construction Materials: Road and Paving Materials; Vehicle-Pavement Systems, ASTM, Philadelphia, PA.

Austroads (1998). Guide to Stabilisation in Roadworks, Austroads Publication No. AP-60/98, Austroads Inc.

Berg, R.R, Christopher, B.R. and Perkins, S.W. (2000). *Geosynthetic Reinforcement of the Aggregate Base/Subbase Courses of Pavement Structures, GMA White Paper II*, Geosynthetic Materials Association, Roseville, MN, USA, 176 p.

Bhatti, M.A., Barlow, J.A., and Stoner, J.W. (1996). "Modeling Damage to Rigid Pavements Caused by Subgrade Pumping," ASCE, *Journal of Transportation Engineering*, Vol. 122, No. 1, pp. 12-21.

Blanco, A.M., Bowders, J.J., Donahue, J.P. (2003). “Laboratory and In-Situ Hydraulic Conductivity of Pavement Bases in Missouri,” presented at the Subsurface Drainage Session, TRB Annual Conference, TRB, Washington, D.C.

CALTRANS (1995) *Design Criteria for Portland Cement Concrete Pavements (PCCP)*. Design Information Bulletin No. 80., California Department of Transportation, Sacramento, CA.

Carpenter, S. H., M. I. Darter, and B. J. Dempsey (1979) “Evaluation of Pavement Systems for Moisture-Accelerated Distress.” *Transportation Research Record 705*, Transportation Research Board, Washington, D.C.

Cedergren, H.R. (1987) *Drainage of Highway and Airfield Pavements*, Robert E. Krieger Publishing Co. Inc. Malabar, FL

Cedergren, H. R. (1988) “Why All Important Pavements Should be Well Drained.” *Transportation Research Record 1188*, Transportation Research Board, Washington, D.C.

Christopher, B.R. and V.C. McGuffey (1997). *Synthesis of Highway Practice 239: Pavement Subsurface Drainage Systems*, National Cooperative Highway Research Program, Transportation Research Board, National Academy Press, Washington, D.C.

Christopher, B.R. (2000) *Maintenance of Highway Edgedrains*, National Cooperative Highway Research Program, Synthesis of Highway Practice 285, Transportation Research Board, National Academy Press, Washington, D.C., 2000, 62 p.

Christopher, B.R., Zhao, A., Hayden, S.A. (2002) “Geocomposite for Pavement Subsurface Drainage,” Presentation at TRB Committee A2KO6 on Subsurface Drainage. 2002 Annual Transportation Research Board Meeting, Washington, DC; also see: Christopher, B.R., Hayden, S.A., and Zhao, A., (2000). "Roadway Base and Subgrade Geocomposite Drainage Layers," *Testing and Performance of Geosynthetics in Subsurface Drainage, ASTM STP 1390*, J.S. Baldwin and L.D. Suits, Eds., American Society for Testing and Materials, West Conshohocken, PA.

Christory, J.P. (1990). “Assessment of PIARC Recommendations on the Combating of Pumping in Concrete Pavements,” Proceedings of the Second International Workshop on the Design and the Evaluation of Concrete Pavements, Siguenza, Spain.

Coduto, D.P. (1999) *Geotechnical Engineering: Principles and Practices*, Prentice Hall, Upper Saddle River, New Jersey.

Daleiden, J. (1998) *Video Inspection of Highway Edgedrain Systems*, Final Report, Brent Rauhut Engineering.

Dawson A R and Hill A R. (1998) "Prediction and implication of water régimes in granular bases and sub-bases", Proc. Int. Symp. On Subdrainage in Roadway Pavements and Subgrades, Granada, Spain, pp 121-128.

De Beer, M. (1990). "Erodibility of Cementitious Subbase Layers in Flexible and Rigid Pavements," 2nd International Workshop on the Theoretical Design of Concrete Pavements, Siguenza, Spain.

Dempsey, B.J. (1982). "Laboratory and Field Studies of Channeling and Pumping," Transportation Research Board, *Transportation Research* Record 849, pp. 1-12.

Elias, V., Welsh, J., Warren, J., Lukas, R., Collin, J.G., and Berg, R.R. (2004). *Ground Improvement Methods*, Participant Notebook, NHI Course 132034, FHWA NHI-04-001, National Highway Institute, Federal Highway Administration, Washington, D.C., 1022 pp.

ERES Consultants, Inc. (1999). *Pavement Subsurface Drainage Design*, Participants Reference Manual for NHI Course Number 131026, National Highway Institute, Federal Highway Administration.

Fehsenfeld, F. M. (1988) *Performance of Open-Graded "Big Rock Mixes" in Tennessee and Indiana*, National Asphalt Pavement Association, Riverdale, MD.

FHWA HI-95-038 (1998 revision), *Geosynthetic Design and Construction Guidelines,* Participant Notebook, NHI Course No. 13213, FHWA Publication No. FHWA HI-95-038 (revised), authors: Holtz, R.D., Christopher, B.R., and Berg, R.R., Federal Highway Administration, Washington, DC.

FHWA (1992) *Drainable Pavement Systems—Participant Notebook.* (Demonstration Project 87). Publication No. FHWA-SA-92-008. Federal Highway Administration, Washington, DC.

FHWA (1994) *Drainable Pavement Systems — Instructor's Guide*. (Demonstration Project 87). Publication No. FHWA-SA-94-062. Office of Technology Applications and Office of Engineering, Federal Highway Administration, Washington, DC.

Hagen, M.G., and G.R. Cochran (1995) "Comparison of Pavement Drainage Systems, Final Report," Minnesota Department of Transportation, Minneapolis, MN.

Hindermann, W. L. (1968) *The Swing to Full-Depth*. Information Series No. 146, The Asphalt Institute, Lexington, KY.

Holtz, R.D., Christopher, B.R., and Berg, R.R. (1998 revision). *Geosynthetic Design and Construction Guidelines,* Participant Notebook, NHI Course No. 132013, FHWA Publication No. FHWA HI-95-038 (revised), Federal Highway Administration, Washington, D.C.

Holtz, R. D., and Kovacs, W. D. (1981). *An Introduction to Geotechnical Engineering*, Prentice-Hall, Inc., Englewood Cliffs, NJ.

Kandhal, P.S. and R.B. Mallick (1997) *Pavement Recycling Guidelines for State and Local Governments*. Federal Highway Administration Publication No. FHWA-SA-98-042, Washington, D.C.

Kaplar, C.W. (1974). "A Laboratory Freezing Test to Determine the Relative Frost Susceptibility of Soils," *Technical Report TR 250*, Cold Regions Research and Engineering Laboratory (CRREL), U.S. Army Corps of Engineers, 1974.

Koerner, R.M. (1998). *Designing with Geosynthetics*, 4th Edition, Upper Saddle River, NJ, Prentice Hall.

McCallister, L.D. and Petry, T.M. (1990). *Property Changes in Lime-Treated Expansive Clay Under Continuous Leaching*, U.S. Army Corps of Engineers, Washington, D.C., No. GL-90-17.

McKeen, R.G. (1976). Design and Construction of Airport Pavements on Expansive Soils. Report No. FAA-RD-76-66.

NCHRP 1-37A (2004). *Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures*, Final Report, National Cooperative Highway Research Program NCHRP Project 1-37A, Transportation Research Board, National Research Council, Washington, D.C.

NHI 131033 **Hot Mix Asphalt Pavement Evaluation and Rehabilitation**

NHI 131060A **Concrete Pavement Design Details and Construction Practices**

NHI 13126 (Currently 131026) (1999) *Pavement Subsurface Drainage Design*, Participants Reference Manual, prepared by ERES Consultants, Inc, National Highway Institute, Federal Highway Administration, Washington, D.C.

PIARC Technical Committee on Concrete Roads (1987). "Combating Concrete Pavement Slab Pumping."

Rollings, M.P., and Rollings, R.S. Jr. (1996). *Geotechnical Materials in Construction*, McGraw-Hill, New York, NY.

Sawyer, S. (1995) *Establishment of Underdrain Maintenance Procedure*. Report FHWA/OK95(04), Federal Highway Administration, Oklahoma City, OK.

Seed, H.B. and C.K. Chan (1959) "Structure and Strength Characteristics of Compacted Clays," *ASCE Journal of Soil Mechanics and Foundation Engineering*, Vol. 85, No. 5, pp. 87-128.

Snethen, D.R., L.D. Johnson, and D.M. Patrick (1977). *An Evaluation of Expedient Methodology for Identification of Potentially Expansive Soils*, Report No. FHWA-RD-77-94, Federal Highway Administration, Washington, D.C.

Sowers, G.F. (1979) *Introductory Soil Mechanics and Foundations: Geotechnical Engineering* (4^{th} edition), MacMillan, New York, NY.

Steinberg. M.L. (1998). *Geomembranes and the Control of Expansive Soils in Construction*, McGraw Hill, Inc., New York, NY.

Thompson, M.R. (1970). *Soil Stabilization for Pavement Systems – State of the Art, Technical Report*, Construction Engineering Research Laboratory, Champaign, IL.

Ulmeyer, J.S., Pierce, L.M., Lovejoy, J.S., Gribner, M.R., Mahoney, J P, and Olson, G.D. (2002) "Design and Construction of Rock Cap Roadways: Case Study in Northeast Washington State" Prepared for presentation at the 2003 TRB Annual Meeting, Transportation Research Board, Washington D.C. Also see *Transportation Research Record 1821*.

U.S. Army Corps of Engineers (1992). *Engineering and Design Drainage Layers for Pavements*, Engineer Technical Letter 1110-3-435, U.S. Army Corps of Engineers, Washington, D.C.

USFS (1977) *Guidelines for Use of Fabrics in Construction and Maintenance of Low-Volume Roads*, authors: Steward, J., Williamson, R. and Mohney, J., USDA, Forest Service, Portland, OR. Also reprinted as Report No FHWA-TS-78-205.

Witczak, M.W. (1972). "Relationships Between Physiographic Units and Highway Design Factors," Report 132, National Cooperative Highway Research Program, Highway Research Board, Washington, D.C.

Yu, H. T., K. D. Smith, M. I. Darter, J. Jiang, L. Khazanovich (1998a). *Performance of Concrete Pavements, Volume III—Improved Concrete Pavement Performance*. FHWA-RD-95-111, Federal Highway Administration, McLean, VA.

Yu, H. T., L. Khazanovich, S. P. Rao, M. I. Darter, H. Von Quintus (1998b) *Guidelines for Subsurface Drainage Based on Performance*. NCHRP 1-34 Final Report, National Cooperative Highway Research Program, Washington DC.

CHAPTER 8.0 CONSTRUCTION AND DESIGN VERIFICATION FOR UNBOUND PAVEMENT MATERIALS

8.1 INTRODUCTION

Thus far, this reference manual has described the processes detailing the pre-construction phase – site characterization and design. The last phase of work required to complete a roadway, the construction phase, is the emphasis of this chapter. As such, construction specifications are described, quality control/quality assurance concepts are reviewed, and innovative measures of design verification are provided. Issues surrounding the preparation of the pavement foundation, focused primarily upon cut and fill soil construction, prepared subgrade, chemically stabilized soils, and unbound engineered aggregate layers (base/subbase) are detailed. Lastly, a review of monitoring techniques for the finished product is provided, an important consideration with the current move toward performance specifications and warranties.

Projects come in all shapes and sizes, each presenting unique challenges. For instance, a new roadway alignment could be conceptualized in a flat topography, requiring little earthwork, or conversely, in a hilly or mountainous topography that requires large cuts – excavation and/or rock blasting – and placement of deep fills. In either case, the engineer typically must make do with the local soils and design for the site conditions encountered. Fundamental to the pavement construction is the preparation of the pavement foundation (*i.e.,* the subgrade) to meet the pavement design support requirements. The designer has made assumptions based on the subsurface exploration program as to the support conditions and the field requirements anticipated to meet those support conditions (*i.e.,* the anticipated adequacy of existing support after grading or modifications required to achieve that support). It is now up to construction to achieve these requirements. Proper treatment of the subgrade during construction will assure expedient construction of the pavement section, enhance pavement performance over its life, and ensure that the pavement design intent is carried through in the construction phase (Ohio DOT, 2002).

8.2 SPECIFICATIONS

The development of the specifications is usually conducted in the pre-construction phase as part of the design. The specification dictates the quality for the pavement section construction, with the intent of tying the design to the finished product (design intent ⇔ performance). There is no good practice other than what is specified in the contract.

Most agencies have developed as part of the construction process a set of standards and specifications. These documents may contain guideline specifications (*e.g., AASHTO Standard Specifications for Transportation Materials*, AASHTO, 2004) modified by the local agency to address local conditions, materials available, construction techniques commonly employed, and their local experience.

The specifications can generally be categorized as *method, result, or performance* specifications. An example of each type is provided below:

Method Specification – This form of specification describes in detail the equipment and procedures (process) used to achieve a desired result (*e.g.,* a compactive effort of 4 passes using a 35-ton sheepsfoot roller – Caterpillar C825 or equivalent – shall be made on each 8-inch lift of loose soil spread on the grade).

Result Specification – The result specification is normally shorter and easier to write than the method specification. This form of specification states what property must be achieved, allowing the contractor some liberty to innovate the process to satisfy the intended result (*e.g.,* a dry density of 95% of the maximum dry density – as determined by AASHTO T99, standard Proctor – shall be obtained for each lift of soil placed on the grade).

Performance Specification – A specification for key materials and construction quality characteristics that have been demonstrated to correlate significantly with long-term performance of the finished work (*e.g.,* the pavement shall support 1 million ESALs without developing fatigue cracks or rut depths exceeding 6 mm (0.25 in.)).

Performance specifications may be presented in one of three forms, including (after Chamberlin, 1995)

- *Performance specifications* – which directly define the condition of the road, the response of the road to load, and/or the condition of the pavement materials at a given period of time,
- *Performance-based specifications* – which describe desired levels of fundamental engineering properties that are predictors of performance; or
- *Performance-related specifications* - which describe the desired level of key materials and construction quality characteristics that have been demonstrated to correlate significantly with long-term performance of the finished work.

The main intent of each type of specification with respect to geotechnical factors is to confirm the adequacy and/or improve the engineering behavior of the soil or aggregate material by modification of moisture content and densification, or compaction, of the soil or aggregate. While the *result specification* is more common, the *method specification* can be utilized where the result is probable based on local experiences, or where the result is difficult to measure (*i.e.,* density of coarse rock fill). This form of specification takes responsibility away from the contractor and places it on the shoulders of the owner and his engineer. The *result specification* will typically encourage the contractor to utilize the most efficient and economical means to achieve the requirements.

Pavement *performance specifications* may be appropriate for design/build and warranty contracts. However, it is obvious that the above pavement *performance specifications* cannot be used to control the quality of aggregate or subgrade materials used in the construction. The pavement is an interdependent layered system consisting of different materials, all of which affect performance. During the service life of the pavement, the material properties can change from those measured during construction. The performance required by the example above is also affected by the thickness of the layers, which is a design element. The main challenge with performance specifications is the determination of performance measures, as discussed later in Section 8.5.

No specification type can cover all situations, and each type has relevance depending on the circumstance (*e.g.,* Design Build or Design-Build-Let contracting methods). The specification, regardless of whether *method*, *result,* or *performance*, should emphasis material properties of raw materials (soil classification, limits for maximum particle size, grain size distribution, Atterberg limits, and other properties typically used for aggregates in base or subbase layers, such as toughness (durability) and soundness, among others).

Each specification type should contain a provision for corrective action measures to be taken when unsuitable conditions (*i.e.,* weak, soft, wet, yielding materials) are encountered. The corrective measures should include

- method of detection (proofroll, QC/QA test, etc.),
- depth of anticipated treatment,
- type of treatment (drainage, undercut and replace, installation of geosynthetics, chemical modification/stabilization), and
- quick resolution determining whose responsibility (pay item) it is to implement the corrective measure (Owner or Contractor).

Once established, site preparation, excavation, hauling, placing, compaction, and grading objectives can commence.

8.3 QUALITY CONTROL AND QUALITY ASSURANCE

Good quality control/quality assurance (QC/QA) practices are essential to obtain satisfactory results in a construction project. QC/QA can be a single plan developed by the Owner to review the construction process. A third party or the agency often performs the quality control (QC) field observations. Alternatively, the quality control (QC) may refer to a written plan submitted by the contractor, which is reviewed and approved by the owner/engineer. This document clearly demonstrates how the contractor will control the processes used to produce or purchase materials used in construction, as well as control the processes for proper installation in order to meet the requirements set forth by the owner/engineer. The QC Plan will typically include tests (QC tests) performed on the materials intended for use at a prescribed frequency, as summarized in Table 8-1, as well as tests to indicate that the intent of the specification is being satisfied (field compaction monitoring and control, again at a prescribed frequency). Quality assurance (QA) is documentation that the contractor is following the QC Plan, and most likely will consist of some random inspections and testing to verify QC observations and results.

Table 8-1. Typical material properties measured for construction.

Test	Frequency
Material Source(s) [1]	
Classification	1 per material type
Atterberg Limits	1 per material type
Grain Size	1 per material type
Moisture-Density (Proctor)	1 per material type
Abrasion [2]	1 per material type
Soundness [2]	1 per material type
Field Installation	
Moisture Content	per QC Plan [3]
Density	per QC Plan [3]
Stiffness Assessment (*e.g.,* proof rolling)	per QC Plan [3]

NOTES:
1: different natural (in-situ or borrow) soil or quarry aggregate.
2: values typically required for quarry aggregate used as base.
3: frequency intervals identified in the QC Plan.

8.3.1 QC/QA: Tradition Methods

The construction specification establishes the framework for QC/QA. With a *method specification*, the quality control (QC) individual would document the equipment utilized and continuously monitor its activities during operation. The assurance may be by certification of QC tests and reports along with intermittent inspection. With a *result specification*, the QC individual would perform frequent testing at the start of the process, testing for changed conditions, and some testing for verification. The assurance testing would typically be a prescribed number of tests for a specific quantity of materials at random locations. Statistical processing of the test data may be used to determine the amount of payment if pay factors are included in the contract. A good practice for quality control is the development and use of a checklist for monitoring and inspecting the construction of the pavement system, similar to the one shown in Table 8-2.

Initial observations include confirming that clearing and grubbing operations have been adequately accomplished and that the prepared surface is suitable for placement of embankment/fill. The "suitability" is often confirmed through proof rolling.

Proof Rolling

The objective of proof rolling is to point out soft or yielding material. The technique can be implemented at any point during construction of the embankment, preparation of the subgrade (top 300 mm (12 in.)), and completion of base and/or subbase layers. In fact, as described later in Section 8.4, proof rolling observations can be made as material is being excavated, hauled, placed, and compacted using the equipment used to perform each of these tasks.

Many agencies have developed vehicle configuration specifications, including weight and tire pressures, for performing proof rolling operations, and have established a policy on methodology and threshold criteria for acceptable deflections, as well as those requiring remediation. For example, Ohio uses proof rolling for all projects types (new, rehabilitation, and reconstruction). This practice is good for detecting soft zones that may have passed the density requirements of the project, but not necessarily the moisture content, and can detect problems that could extend many feet below the tested surface. Once detected, seasoned experience can often estimate the depth of probable weakness; however, penetration rods and hand augers can be used with more objectivity than the eye guestimate. Once detected and properly delineated (aerial extent and depth), remediation actions are typically employed (remove/undercut and replace, installation of underdrains, installation of geosynthetics, chemical stabilization) that best suit the conditions encountered. The remediation alternative selected typically is a result of a cost or schedule constraint. Many agencies have reported historically large change order work dealing with soft subgrade, and have subsequently included likely remediation alternatives in the bidding process to establish a competitive rate for this work.

Table 8-2. Field monitoring checklist.

1. Read the specifications and become familiar with

- site preparation
- material requirements
- construction procedures
- grade/slope requirements
- drainage requirements
- tolerances of each of above requirements
- testing requirements
- acceptance/rejection criteria

2. Review the construction plans and become familiar with

- lines, grades, and layer thickness requirements
- temporary and permanent drainage features, locations, and details
- details for utility construction, special requirements
- demolition (if rehabilitation or reconstruction project)
- corrective action requirements for weak, yielding, unstable materials (undercut/replace, chemical stabilization, geosynthetics)
- construction sequence

3. Review material requirements, equipment requirements, and approved submittals.

4. Check site conditions. Observe

- clearing, grubbing requirements, and activities – document the final condition in accordance with the specification
- haul patterns
- response to load (deflection under heavy equipment traffic)
- assess need for drainage system
- perform foundation acceptance – testing as required (*e.g.,* proof rolling, DCP, etc.)

5. On site monitoring and testing.

- observe and document hauling, placement, and spreading adequacy (segregation, loose lift thickness, moisture content reasonableness)
- review truck tickets and ensure that material sources are from approved sources (generally for engineered products such as aggregate base or flowable fill, bedding material, and off-site borrow, etc.)
- randomly sample for material compliance
- assess for moisture content reasonableness (not overly wet or dry in conformance with the specification)
- document compaction efforts (size and type of equipment, # of passes)
- test for compaction (determine conformance to specifications with respect to density and moisture – Proctor check point, if necessary) and assess stability
- non-conformance corrective action – proof roll and assess extent/depth of affected area; determine and document responsibility (owner vs. G.C.) and begin corrective action
- measure and document stabilizing agent type and rate of application, mixing adequacy and depth, compactive effort, moisture reasonableness and compaction compliance results (for stabilization/modification layers or treatments). Sampling and testing of treated layers and determination of strength in the laboratory may also be required.

Tests

Test methods used for in-place quality control and acceptance of individual flexible pavement layers and of new and rehabilitated flexible pavement systems have changed little in past decades. Such quality control and acceptance operations typically rely on nuclear density measurements (Figure 8-1), sand cone, balloon or drive tube methods, and the results of moisture content determined by a variety of methods, including nuclear gauge, speedy moisture, and hot plate or oven drying. These tests are typically performed on embankment construction (fill soils), finished subgrade, and unbound base layers, while some are applicable to measuring the quality of chemically stabilized materials, as described later in this section.

The old school of thought used compaction testing to calibrate construction methods. After the methods were calibrated, observation became as important as testing for quality control. Samples were taken at select locations based on observations. Today there is more of an emphasis on statistical characterization of constructed materials. Sample locations have become more random. Quality assurance specifications often give the contractor the responsibility of sampling and testing for process control. Testing by the owner includes some verification of the contractor's test results, and testing for acceptance and payment. The amount of payment may be determined by the statistical evaluation of test values resulting in pay factors (and no test reports, no pay).

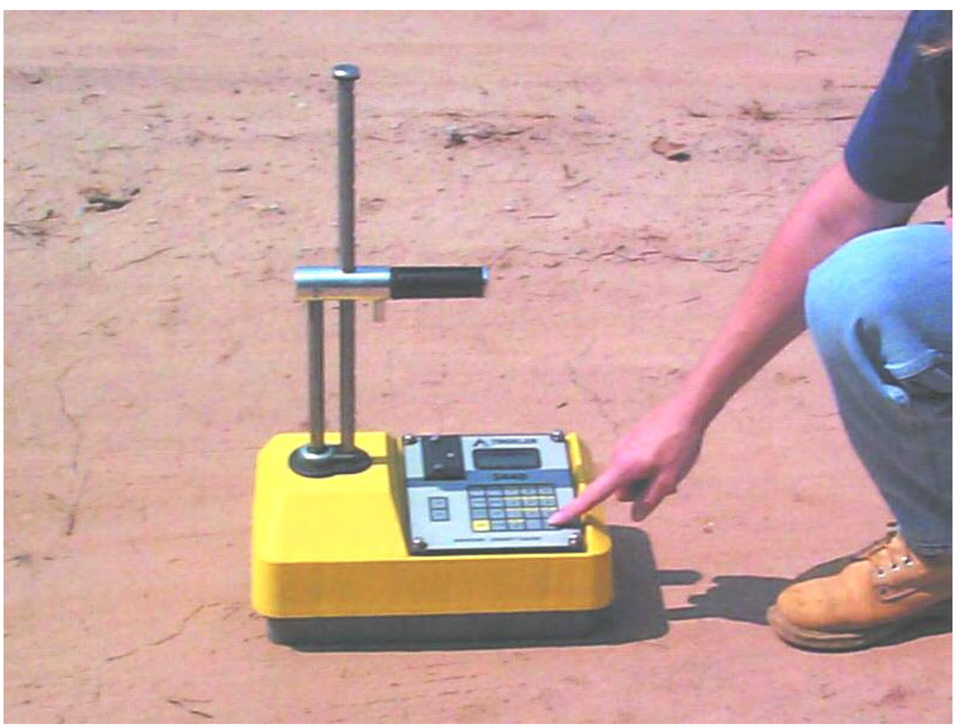


Figure 8-1. Nuclear densometer (*photo courtesy of Troxler)*.

As part of a good QC Plan, process control and measurement of the control can be a valuable tool. A test result, or trend of a measured value, may not directly demonstrate compliance or non-compliance, but tracking the measured value over time may help explain why another process is out of compliance. In the following example, a soil with a standard proctor maximum dry density of 15.7 kN/m^3 (100 pcf) at an optimum moisture content of 20% is being placed in a single lift along a 300-m (1,000-foot) length of roadway embankment. The specification requires that a minimum of 95% of the maximum density be obtained, at or near the optimum moisture content. In order to simplify this example, it is assumed that the material is uniform in classification, and is being hauled by scrapers from a cut zone nearby. QC tests have been recorded and are graphically shown in Figure 8-2. This figure illustrates that the density is adequate along the first 120 m (400 feet) of placement, then trends toward an 'out-of-tolerance' or 'out-of-control' situation. The QC Plan may prompt the contractor to exert more compactive effort on this 'out-of-control' area, or change compactors; however, the moisture data suggests that the moisture content may be the 'out-of-control' parameter, which is, in turn, causing the density to move 'out-of-control'. By recognizing what part of the process is defective, the contractor begins spreading the cut soil in thin lifts, allows some drying to occur prior to compacting, and again returns to a product considered satisfactory at the 250-m (820-foot) mark.

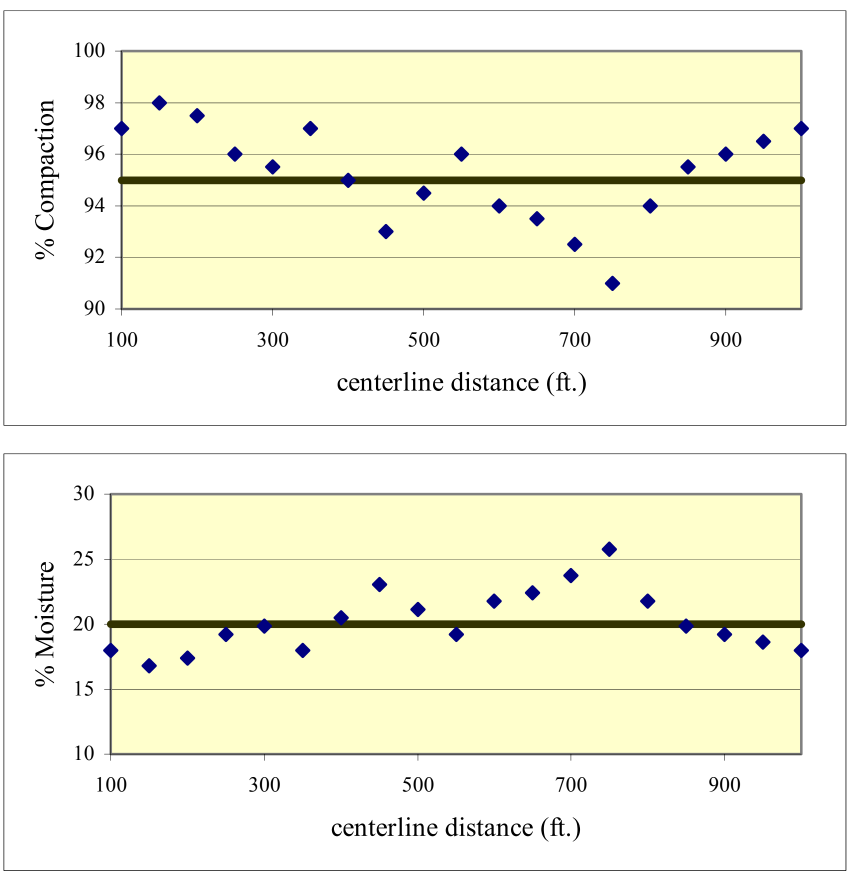


Figure 8-2. Process control, field density and moisture. (3.3 ft = 1 m)

This form of process control (density or degree of compaction) is applicable to embankment, subgrade, and unbound aggregate base construction. In addition, gradation of aggregate base materials is an important process control measurement. Depending on location of sampling (on grade, from haul trucks, from stockpiles) segregation and contamination may be detected using this measure. It is also an important measure used to ensure that the quarry process (crushing and grading) is in control.

Chemical Stabilization

In addition to density and moisture measurements, several additional controls are required and considered good practice when stabilization techniques are employed. An excellent source for QC/QA requirements can be found in the Soil Stabilization for Pavements manual (Army, Air Force, 1994). Briefly, these elements include

- Pulverization and scarification – An assessment of the material to be treated is required, and generally includes random sampling and testing using a field sieve (25-mm (1-in.) and 4.75-mm (No. 4) sieves).
- Stabilizing agent content – The amount of modifier added to the soil should be measured, and generally includes sampling the discharge using a canvas of known area placed on the soil to be treated, or calculating the area over which a known tonnage has been spread. For lime stabilization, pH can provide a good indication that the correct dosage has been achieved, as discussed in Appendix E.
- Uniformity of mixing – Visual observation is made to ensure that uniform mixing has been accomplished throughout the full depth of treatment. The use of phenolphthalein indicator solution has been used effectively for lime treatments to indicate depth of treatment. This solution – a light spray applied to the sides of a hand-excavated hole in the treated soil – will react with the lime, turning a brilliant pink color.
- Compaction and moisture control – Covered in the previous section.
- Curing – Curing is essential to assure that the modified soil mixture will achieve the final properties desired. The use of moist curing (light sprinkling of water) or membrane curing (application of a bituminous coating) is common. Regardless of method, the entire area must be properly sealed, and documentation of this activity is required.

Again, most QC programs only measure the compaction of the earthwork operation. While this methodology is valuable, density does not necessarily translate into performance. This type of QC testing is still very important in that it provides for a uniformity in the contractor's work, and can control the moisture of a given soil type at or near its optimum moisture content. This moisture control is important in order to minimize volume change characteristics, and none of the technologies described above have this capability.

8.3.2 QC/QA: Emerging Technologies

Lost in most projects requiring earthwork operations is the *design intent*. While the industry has accrued decades of experience founded upon successful engineered and constructed projects, we still observe the occasional premature failure that could have possibly been avoided. Reviewing, site soils are improved for three reasons:

- there are large quantities available;
- the natural state is inadequate to support the intended structure; and
- it is cost-effective.

For embankment fills, shear strength (slope stability) and consolidation (settlement potential) are used in the engineering analyses for *design intent*. As the elevation gets closer to the final grade (below the granular layers and surface layers – or within the anticipated depth or zone of repeated loading influence) resistance to deformation – siffness or resilient modulus – is used in the engineering analyses for *design intent*. These parameters are measured in the laboratory on soils sampled in the soil exploratory phase of the design. In essence, the design parameters are rationalized as an anticipated, educated estimate of what will finally be obtained in the field. But rather than measure these design parameters in the field, we commonly accept the contractor's work based on a *result specification* measurement of density. While this may be common and traditional, it does not verify if the *design intent* has been met.

As an example, the following scenario represents a hypothetical design situation where a 0.4-km (¼ -mile) roadway, 2 lanes wide is to be constructed along a gently rolling topography. Based on soil borings drilled and samples obtained, classified and tested, Table 8-3 was compiled for five uniquely different soils (based on visual description and engineering classification) ranging from a sandy soil to a highly plastic clay. The five samples are identified as Samples A-E.

In this hypothetical example, it is assumed that each of these materials will be equally represented along the length and width of the project, and are semi-infinite in depth. Further, the design assumes each material will be field compacted to 95% of the soil maximum dry density at or very near its optimum moisture content. Lastly, it is assumed that the soil mass will remain at this condition for the performance period. Each of these assumptions is idealistic, not realistic. However, it can be demonstrated that the *design intent* for this hypothetical example can be verified in the field during construction. In this example (Boudreau, 2003), the *design intent* is a roadbed stiffness of 20 Mpa (7200 psi) based on a stress state of 36 kPa (5.3 psi) vertical and 11 kPa (1.6 psi) horizontal (as estimated by the pooled subgrade constitutive model $M_r = 9041\sigma_v^{-0.19526}\sigma_h^{0.19643}$ with σ_v, σ_h in English units).

Table 8.3. Summary of design soil properties for example problem (pre-construction).

Physical Property		A	B	C	D	E
Liquid Limit, LL		21	NL	35	64	36
Plastic Limit, PL		16	NP	14	29	27
Plasticity Index, PI		5	---	21	35	9
P_4 (%)		94	100	100	100	100
P_{10} (%)		92	100	100	96	96
P_{200} (%)		47	20	59	82	48
Maximum Dry Density,[1] γ_{max}	(kN/m^3)	18.8	18.2	16.9	14.9	17.8
	(pcf)	(119.8)	(115.9)	(107.8)	(94.7)	(113.3)
Optimum Moisture Content, w_{opt} (%)[1]		12.0	11.8	17.2	25.6	15.0
AASHTO Classification		A-4	A-2-4(0)	A-6(9)	A-7-6(32)	A-4
Unified Soil Classification		SC	SM	CL	CH	SC
Resilient Modulus Parameters[2]	K1	10 387	6246	10 274	10 362	7938
K2		-0.015483	-0.00836	-0.41797	-0.18345	-0.21171
K3		0.23229	0.30028	0.08425	0.12762	0.23770

1. Maximum dry density and optimum moisture content, as determined by AASHTO T-99 (standard Proctor).
2. For modulus equation: $M_r = K1 S_V^{K2} S_3^{K3}$ with S_V and S_3 in English units. Laboratory test specimens prepared to 95% of maximum dry density at optimum moisture content (as determined by AASHTO T99).

Based on the information provided, a conventional pavement cross section resulting from the subgrade support conditions, determined from the pre-construction laboratory test program summarized in the table above (analyses performed per AASHTO 1993 Design Guide using estimates for traffic and other inputs per the Boudreau reference cited) is 140 mm (5.5 in.) of asphalt concrete on 200 mm (8 in.) of a crushed aggregate base. The question is, if the contractor satisfies the *result specification* for subgrade construction - for this example, the contractor must achieve 95 percent compaction at or very near optimum moisture content - and the layers above (140 mm (5.5 in.) of asphalt concrete and 200 mm (8 in.) of a crushed aggregate base) are constructed with approved materials and constructed to satisfy the *result specification* for these layers, will the pavement perform as designed and expected? The answer deserves some examination.

First, the *design intent* for the subgrade is stiffness and strength; the measure of acceptance is density. Fundamentally, these two measures are uniquely different; one measure does not necessarily confirm the other measure. It is possible that the contractor has met the compaction specification on the wet side of optimum. The important measure is one of stiffness – in this hypothetical example, 50 MPa (7200 psi).

There are a number of ways to more precisely measure whether the *design intent* has been satisfied. These measures could include field CBR and plate load tests, dynamic cone penetration (DCP) tests, correlation studies, and/or laboratory tests performed on undisturbed tube samples obtained at finished grade. More recently, nondestructive testing (NDT) methods, including lasers, ground-penetrating radar, falling weight deflectometers (FWD), mini or portable lightweight FWD (LWD) cone penetrometers, GeoGauge (providing direct stiffness measurements), and infrared and seismic technologies, have been significantly improved and have shown potential for use in the quality control and acceptance of flexible pavement construction. As mentioned in Section 8-4, another technology in development consists of instrumented compaction equipment. This and the others mentioned above require field verification studies prior to any endorsement of the technology. The thrust of NCHRP Project 10-65 is to explore many of these technologies for this specific application (Von Quintus et al., 2004). It is anticipated that some of these techniques will eventually be incorporated into *performance specifications* as the industry gains more knowledge and accrues more experience with them. Many of these techniques were briefly reviewed in Chapter 4 (see Tables 4-2 through 4-6) and are described in greater detail below.

With the advent of the much anticipated NCHRP 1-37A Pavement Design Guide and extended warranty period of performance, there is an ever increasing need to measure layer stiffness properties by owner agencies, an activity that is not presently a typical component in the acceptance of a completed project.

Proof Rolling

A practical approach that many agencies use is the concept of proof rolling, as discussed in the Section 8.3.1. Although this approach is observer-dependant, many agencies use the technique not to measure *design intent* (deformations anticipated at stress levels typical under repeated load traffic protected by layers of material would result in deformations undetectable to the human eye during a proof rolling exercise), but to evaluate gross deficiencies including soft, yielding, or pumping subgrade. The objective of this type of process is to correct problem areas prior to the placement and compaction of stronger, more expensive materials (these soft zones will surely be detected during finishing operations of the stronger layer materials in the form of roller cracks).

With newer more sophisticated technology, including lasers, digital video, and image analysis, it is possible to take proof rolling to a new level of direct stiffness measurements. Small deformations can now be monitored as the proof roller moves across the site. Although this is somewhat of a research topic at this time, the concept is fairly straightforward to develop. In fact, Wisconsin DOT has developed a prototype deflection measurement system for use with a loaded dump truck, using ultrasonic sensors and a micro-controller, in order to continuously and objectively proof roll subgrade soils. Wisconsin DOT concluded that a threshold value of 38 mm (1.5 in.) of deflection indicated “failed” areas that required corrective action, and also found value in analyzing the ratio of the 0-offset and 0.6 m-offset (24 in.) sensors to determine depth of weakened zones (Wisconsin DOT, 2002).

Field CBR or Plate Load Tests

These technologies were developed several years ago and were employed as a measure for verifying *design intent*. Each included mobilization of equipment (moderate to heavy plates, loading rams, calibrated proving rings or load cells, and dial indicators or electronic deflection measurement devices) crew and heavy reaction vehicle (typically readily available on an earthwork construction project in the form of a scraper or track-mounted excavator/shovel). These tests are often the standard for quality programs in Europe, but have not typically been utilized in the U.S., based on their relative cost, time involved to set up and perform the test at a specified location, and accuracy issues. The field CBR test could measure the in-situ CBR of finished subgrade in order to verify *design intent* for flexible pavements, and the plate load could directly measure the modulus of subgrade reaction, or k-value, for rigid pavements. The plate load test is the standard practice in Europe for all pavement types.

Each of these measures is time-consuming; thus, only a few locations could be tested per day, oftentimes impeding the earthwork contractor's progress. In cut zones, these tests measure soil properties that are not controlled by the contractor, thus it is often difficult to expect the contractor to achieve a predetermined CBR or k-value threshold without paying for corrective measures.

It is noted that this type of testing is common and traditional in many European countries, using special customized equipment to make this type of testing more automated and more productive than that described above.

Dynamic Cone Penetration (DCP) Tests

The DCP technology consists of a steel shaft with an instrumented penetration head conforming to a precise configuration, as was described in Table 4-9. The instrumentation is capable of measuring resistance per increment of advancement and used with correlations to estimate stiffness of the materials. The benefits of this form of measurement are that the device can be quickly and efficiently mobilized to the project site (can be hand-carried or mounted inside a vehicle) and can measure to depths beyond surficial soils. The drawbacks include discrete point evaluations – leaving zones between points unknown, and the fact that the information gathered is used to correlate stiffness and strength. Thus, is only as accurate as the correlation models used. However, an added value is that the DCP can readily indicate soil support via correlations for construction activities. For example, if the estimated in-situ CBR has a value of 6 or less, the soils are expected to rut and deflect under construction operations. If the estimated CBR values are between 6 and 8, the soils are considered marginally suitable for construction support (Illinois DOT, 1982).

Resilient Modulus Testing

The design for the example introduced at the beginning of this section was based upon several soil samples characterized in the laboratory. The soils selected for characterization were those anticipated in the uppermost zones of the finished subgrade. In order to verify *design intent*, it would seem logical that samples of the earthwork contractor's finished work be sampled and characterized in a similar fashion. This can be accomplished by extending short Shelby tubes into the compacted soil and returning to the laboratory for resilient modulus testing. The testing can occur on extruded tube samples the same day they are obtained from the field. Thus, final reporting can be available the next day. This form of sampling and testing has the benefit of comparing actual results with those used for design purposes. Additionally, these measures are direct; therefore they are not reliant upon correlations. Lastly, because these samples are physical in nature, the density and moisture content will be measured and can be compared with the QC test results for accuracy of the QC testing program.

Falling Weight Deflectometer (FWD) Tests

A very mobile device and one that can be utilized to examine the stress dependency of the embankment or roadbed soils, a falling weight deflectometer is basically a trailer-mounted piece of equipment, which drops a weight transmitted through a hard rubber-type pad to the surface (as covered in detail in Table 4-2). The van pulling the trailer is equipped with a computer data acquisition system that measures the load and offset surface deflections. For field control, there are also portable or lightweight LWD units (as shown in Figure 8-3), allowing an individual to carry the unit around in any vehicle.

This technology, with sophisticated computer models, can directly measure the roadbed deflection from which modulus values can be estimated in order to verify *design intent*. The device is relatively quick (less than 4 minutes is required per location to measure the properties), thus numerous locations can be measured per hour. There are also several new developments with units mounted on sleds or skis such that continuous coverage along the length of a project is possible.

Figure 8-3. Lightweight deflectometer (*photo courtesy of Dynatest*).

GeoGauge

A recent development is the GeoGauge, a lightweight unit capable of measuring stiffness at discrete points. The Federal Highway Administration is currently evaluating this technology in the form of a Pooled-Fund Study. This device has many perceived benefits, including the capability to measure the stiffness of a composite soil mass directly and quickly such that numerous discrete points can be evaluated per hour.

Seismic Methods

The Portable Seismic Pavement Analyzer (PSPA) and a derivative modified for base and subgrade measurement, the Dirt Seismic Pavement Analyzer (DSPA), are currently being used on a trial basis by the Texas Department of Transportation for QC/QA purposes (Nazarian, 2002). The operating principal of the PSPA is based on generating and detecting stress waves in a medium. If used appropriately, analyses of the stress waves can be used to determine the modulus of the layered material, as well as assess the thickness of the layer (aggregate base). These techniques are being utilized with very promising results during construction on a few projects in Texas, and are being considered for quality control on pavement warranty projects in Texas and New Mexico. This method of measurement and analysis are very similar to the principals used in spectral analysis of surface waves (SASW, Table 4-6).

Automatic Controlled Variable Roller Compaction and Documentation System (Intelligent Compaction)

While each of the aforementioned techniques have perceived advantages and disadvantages, none of the techniques described above has the capability to continuously evaluate *design intent* along the entire length of a project. The use of instrumented compaction equipment would appear to have some potential for continuously monitoring conditions along a length of a project.

The real-time automatic controlled variable roller compaction and documentation system (a.k.a. intelligent compaction) allows for optimization of compaction rates and real-time quality control. The system works by using accelerometers to monitor the speed of the dynamic wave through the soil, induced by the vibratory rollers, in order to measure the dynamic stiffness of the soil, which generally increases with higher compaction. Efficient fill densification is achieved via automatic adjustment of compaction energy and the measurement/documentation feedback, eliminating time wasted on compacting areas that are already adequately compacted. This energy variability and efficiency is achieved by the use of two counter rotating weights in the drum, rather than the conventional single, one-directional eccentric weight. The weights rotate in opposite directions and only come together in a common direction in the downward vertical inclination. This eliminates unwanted and wasteful movements in the lateral and upward directions that occur with conventional compaction drums. Internally, the entire counterweight assembly is rotated to adjust the direction of the point where the two weights act together. If the onboard monitoring system determines the soil is compacted to a satisfactory level, it will automatically reduce the vertical component of force at the specific time and location.

In addition, the ability to monitor density improvement during compaction both speeds up and improves the aerial extent of quality control. Most importantly, the ability of instrumented compaction equipment to provide 100% quality control coverage enables the use of performance based approaches to specifications, and the effective implementation of warranties and guarantees for both earthwork and pavements.

This method does require proprietary specialized monitoring equipment, but the equipment and process are not patented. The equipment is readily available in the U.S., and requires nominal operator training.

8.3.3 Risk Acceptance

Warranties for materials and workmanship are common in the construction industry, with most performance bonds covering such items for 1 year following completion of a project.

However, the new emphasis on warranties for highway construction involves the guarantee of the long-term performance of highways. Typically, a long-term warranty is considered to cover a period from 2 – 5 years. It is beyond the generally accepted standard warranty period of 1 year. This creates a very difficult situation, and one that involves a very high degree of uncertainty with our current state of practice and technology in the United States highway construction industry (Hancher, 1994 - NCHRP Synthesis 195).

This shift in philosophy has basically brought about a shift in project risk. Traditionally, the owner has assumed nearly all the performance risk by developing the design, specifying materials, and either specifying the results (density) or method to achieve the desired result, and measuring and accepting the contractor's work. The contractor is at risk for gross failures resulting from noncompliance with contract requirements detected in the first year of performance, while the owner assumes responsibility for failures and maintenance following the initial year of performance. By extending the initial warranty period from 1 year to a period up to 5 years, the owner has shifted some of this inherent risk to the contractor. As a result, the contractor has been tasked with becoming a more integral part of the design and construction process.

Extending the period of performance and assigning performance risk to various parties has lead to more sophisticated approaches with respect to life-cycle performance monitoring, early detection of potential problems, cost analyses, and budget optimization. For a contractor to be willing to accept more risk, he or she should have a more active role in design.

8.4 CONSTRUCTION AND CONSTRUCTION MONITORING

Construction of the pavement involves grading to provide a uniform support layer(s) at the appropriate elevation. In modern construction, either major earthwork or reconstruction, sophisticated equipment is available to excavate, haul, add water, aerate (decrease water), spread, and compact to achieve this purpose. The objective of this type of operation is to achieve a structure with specific design intent. Most earthwork projects have the additional objective that transforms existing or natural topography to an acceptable and safe vertical and horizontal alignment. A common goal is to achieve this new alignment with a balance of site materials. Projects that require more soil in fill areas than can be produced from cut zones will require additional materials from off-site borrow sources. Projects that generate more material from cut zones than can be placed in fill areas will be wasted.

The construction process is described in this section, along with the requirements for monitoring each phase of the construction activities, as outlined in Section 8.3 and summarized in Table 8-2.

For new construction, subgrade preparation will typically require grubbing and grading (either cut or fill) to meet subgrade elevation requirements. In either case, clearing and grubbing is very important to remove vegetation, debris, and any organic, soft, or otherwise unsuitable materials from the surface of the site, either at subgrade level or before placing fill. For reconstruction, the old roadway surface will be removed (possibly recycled), possibly along with the base and subbase layers, if they are not suitable for support (*i.e.,* intermixed with large amounts of fine-grained soil). Observation of heavy equipment operations on the site at this phase provides the first indication of the subgrade adequacy. Rutting and deflection during initial earthwork operations indicates an immediate problem. This may not be a problem if significant undercut or stabilization was anticipated prior to construction. However, if soft materials are encountered at subgrade or initial fill elevation and were not anticipated, immediate action should be taken. In either case, the conditions must be improved.

In order to improve the soil conditions at a site, a couple of options are common:

1. Remove marginal soils and replace with select material of higher quality.
2. Stabilize in-place soil (using one of the many techniques discussed in Chapter 7).
3. Improve site drainage.

Often the removal and replacement alternative is used because it appears to be the easiest. This is generally true for small areas or areas where spot locations are identified. However, undercutting may not be the most effective or even desirable for large areas. Excess water in soil is the principal cause of unstable conditions, and reducing the soil water content either by dewatering or stabilization methods may offer a better solution to expedite construction. Hauling or loading on a wet subgrade may continually disturb the section. Excavation in wet conditions often leads to more excavations. The variation of soil types and saturation levels should be evaluated by subsurface investigation to determine the vertical and longitudinal extent of the problem before a decision is made on which method to use. Excavation, drainage, and confirmation of stabilization will be reviewed in greater detail in the next section on construction techniques.

Proper consideration of existing conditions should be given prior to earthwork construction. For example, areas that are to receive fill soils should be cleared and grubbed (*i.e.,* vegetation, organic soils, and weak or otherwise unsuitable soils removed). Long-term performance issues are often traced to inadequate removal of unsuitable materials. Removal of surface soils containing organic matter is important, not only for settlement, but these soils are often moisture sensitive, losing significant strength when wet, and are easily disturbed under construction activities. Site QC/QA personnel should monitor and confirm that these organic soils have been adequately removed. Documentation should including a visual description along with photos of the cleared surface. If additional excavation will not be made, the surface should be checked at this point for compliance with specification requirements. This will often require proof rolling or other testing such as DCP. Unsuitable materials also often find their way back into filling operations. Therefore, the control of unsuitable materials should also be documented. Other special earthworks should also be observed and documented, including embankments properly sloped to prevent slides, keyways (a.k.a. shear keys) constructed to avoid toe slope failures, and erosion protection techniques to maintain long-term stability. Large embankments can cause settlement of the natural soils, thus analyses examining settlement potential of underlying soils is essential. If settlement is anticipated, techniques such as removal-and-replacement or surcharge embankments can be utilized to mitigate the problem (See FHWA NHI-00-045, 2000).

In unusual circumstances, a roadway alignment may be constructed over large voids or weakened soil zones, such as caves, faults or collapsed, abandoned mines, and sinkholes. Detection of these underlying potential problems prior to construction is ideal. Geophysical techniques such as ground penetrating radar (GPR) or spectral analyses of surface waves (SASW), again as described in Chapter 4, may be used in conjunction with historical information and experience to detect or predict the likelihood of encountering these described problems. Mitigation may include excavation and backfill operations, injection grouting, or grout columns using tubular fabric forms. For very localized areas and in karstic regions where voids are random but anticipated to be small, there has also been some success with bridging such areas with thickened reinforced concrete slabs or reinforced base and subbase using geogrids or welded wire mesh. However, for these techniques to be successful, the areal extent of the subsidence area must be clearly defined.

8.4.1 Drainage

If conditions exist during construction that indicate the need for underdrains (*e.g.,* wet, saturated conditions) or the cleaning of the existing underdrains outlets, then this work should start as soon as possible (after Ohio DOT, 2002). Some examples of these conditions are as follows:

1. existing underdrains with clogged outlets on rehabilitation projects
2. free water in the subgrade
3. saturated soils of moderately high permeability, such as sandy silt and silty clay of low plasticity

4. groundwater seepage through layers of permeable soil
5. water seeping in the test pits
6. water seeping from higher elevations in cut locations
7. water flowing on the top of the rock undercuts

Significant subgrade stability improvement can be obtained by cleaning out the existing underdrain outlets on rehabilitation projects and by adding construction underdrains on new construction projects. The FHWA/NHI course manual on Pavement Subsurface Drainage Design is a useful reference. Once the underdrain systems are in place and functioning, the drainage system can typically reduce subgrade pumping problems within a few days, but may take longer depending on the characteristics of the in-situ materials. Soils that are subject to densification and are not free draining (percent saturation exceeding 80 – 90%) within 1 m (2 – 4 ft) of the surface are not expected to support construction traffic. This order of magnitude of saturation is frequently observed to be the limit for compaction stability when developing moisture-density curves in the laboratory. Saturated soils with more than 10% fines (minus 0.075 mm (No. 200) sieve) are not expected to be drainable with respect to supporting construction traffic. Moisture reduction by evaporation (*e.g.,* disking and aeration) may be more feasible than gravity drainage for these types of soils.

For rehabilitation projects, the Contractor should be instructed to unclog the underdrain outlets immediately, attempting to perform this work in the timeframe listed above. If the project consists of several phases, then the Contractor should perform the outlet cleaning for the entire project at the same time. Because of the timeframes involved, construction underdrains should not be used for rehabilitation projects.

For new construction projects, subgrade stability can be achieved by constructing the plan or construction underdrains as soon as the water problem is found (see Figure 8-4). New construction projects can allow a longer period of time for the underdrain system to work. At the beginning of construction, and certainly before winter shut down, are opportune times for this work.

The plan underdrains should be placed only when they will not be contaminated by further construction. If contamination is a concern, then sacrificial or temporary construction underdrains should be used on the project.

Construction underdrains are usually placed in the centerline of the roadway. They may also be placed in the ditch line, if the water is coming in from a cut section at a higher elevation. The porous backfill is extended to the subgrade elevation. The outlets for the construction underdrain are the same pipe material and backfill as regular underdrains. The underdrains can be outlet to any convenient location. Some potential outlet locations are catch basins, manholes, pipes, or ditches. The project should not be concerned with the contamination in the upper portion of construction underdrain backfill. Construction underdrains are sacrificial underdrains that will continue to work throughout the life of the contract and afterwards even though the upper portion is contaminated.

For rock or shale cuts, the design underdrains should extend at least 150 mm (6 in.) into the existing rock formation. If the underdrains are too high, the water will accumulate at the rock and soil interface and cause subgrade instability.

Figure 8-4. Underdrain installation (*photo courtesy of Ohio DOT*).

8.4.2 Excavation

Most construction projects will consist of some amount of excavation, or removal of in-situ soil to some design elevation or grade line. Excavation is typically accomplished using scrapers (also referred to as pans) or shovels, which are among the list of heaviest equipment used in modern earthwork. Observations made at this stage of construction are considered the first line of QC documentation. Site personnel should observe vertical movements below the construction equipment during excavation. Moderate to large deflections are the first indication that weak soils exist and some corrective action may be necessary.

Scrapers have the ability to remove material from grade and spread at another location nearby. Some scrapers may need to be pushed by another scraper or by a bulldozer in order to advance while cutting into the zone of soil to be removed. Other scrapers are equipped with an elevator system (Figure 8-5) that allows the excavated material to be readily loaded without the assistance of a push from behind.

Scrapers are commonly used in cut-fill earthwork operations, where the majority of the soil excavated is placed along another portion of the project. Anticipated site conditions that consist of wet or saturated soils due to water table elevations may necessitate the use of other forms of excavation equipment in order to minimize disturbance to the underlying in-situ soils. A common piece of equipment is the track-mounted excavator (shovel) like the one illustrated in Figure 8-6. Materials removed or excavated by this means require transfer to a secondary piece of equipment for hauling off site, or to another location along the alignment where fill soils are required.

8.4.3 Hauling and Placement

While scrapers (a.k.a. pans) transport their payload from a cut zone to a fill area, shovels require a haul truck to be utilized. There are several types of hauling vehicles, including end dump, side dump, bottom dump (or belly dump), and articulated dump trucks, as illustrated in Figure 8-7.

Figure 8-5. Self-loading scraper (*photo courtesy of Caterpillar*).

Figure 8-6. Track-mounted excavator (*photo courtesy of Komatsu*).

Figure 8-7. Articulated dump truck (*photo courtesy of Komatsu*).

Some projects will require off-site materials to be hauled in because of an imbalance of site materials. These borrow materials will typically be hauled in trucks and dumped near their intended final location. Depending on the dumping method, these piles may require spreading using a bulldozer or motor-grader (shown in Figures 8-8 and 8-9, respectively). Again, observation of this activity can indicate soft or unsuitable areas that will require special treatment. When excessive rutting is noted, haul routes should be changed so as to minimize the depth of disturbance. An assessment by the engineer should be made as soon as practical to determine if underdrains are needed. Often, well-placed and well-timed construction underdrains can mitigate the problem, and hauling over the previously unstable location may improve the stability by adding compaction to the draining soils.

Some projects may restrict hauling on existing paved roads in order to eliminate damage to existing local roadways. In this case, it is possible to utilize a conveyor system to transport the borrow material to the site, like the one shown in Figure 8-10.

During the hauling operation, material to be used as fill should be sampled and tested by QC/QA personnel for compliance with the specification requirements (*e.g.,* soil type, gradation, etc.) Laboratory moisture-density tests (a.k.a. Proctors) should also be preformed for correlation with field density testing.

Figure 8-8. Bulldozer (*photo courtesy of Komatsu*).

Figure 8-9. Motor grader (*photo courtesy of Caterpillar)*

Figure 8-10. Earth-moving conveyor system (*Atlanta Airport – 5th Runway Embankment Placement*).

8.4.4 Field Compaction

Compaction can be defined as the densification of soils by the application of mechanical energy, oftentimes requiring a modification of water content. The purpose of compaction is generally to enhance the strength or load carrying capacity of the material, while minimizing long-term settlement potential. By adjusting the moisture content to a value at or near a moisture content considered optimum – as described below – reduced volume changes and increased strength can be achieved.

Significant advances have been made in the science and technology of earth structures in the last century. In the early 1900s, soils were placed in embankments by end dumping from wagons, with little attempt to compact. Structures that were placed by hand using baskets had, at a minimum, foot traffic to unintentionally "compact" the soil. It was observed that this foot traffic actually strengthened the soil, thus creating the concept of mechanical stabilization. Different types of field compaction equipment are appropriate for different types of soils. Steel-wheel rollers, the earliest type of compaction equipment, are suitable for cohesionless soils. Vibratory steel rollers have largely replaced static steel-wheel rollers because of their higher efficiency. Sheepsfoot rollers, which impart more of a kneading compaction effort than smooth steel wheels, are most appropriate for plastic cohesive soils. Vibratory versions of sheepsfoot rollers are also available. Pneumatic rubber-tired work well for both cohesionless and cohesive soils. A variety of small equipment for hand compaction in confined areas is also available.

Recommended field compaction equipment for various soil types is summarized in Table 8-4. The effective depth of compaction of all field equipment is usually limited, so compaction of thick fills must be done in a series of lifts, with each lift thickness typically in the range of 150 – 300 mm (6 – 12 in.) with greater depths possible (up to 0.7 m (2 ft)) through the use of specialized high energy equipment and the right type of soil conditions (*e.g.,* free-draining granular soils). The soil type, degree of compaction required, field compaction energy (type and size of compaction equipment and number of passes), and the contractor's skill in handling the material are key factors determining the maximum lift thickness that can be compacted effectively. Control of water content in each lift, either through drying or addition of water plus mixing, may be required to achieve required compacted densities and/or to meet specifications for compaction water content.

Proof rolling with heavy rubber-tired rollers is often used to achieve additional compaction beyond that from normal compaction and, more important, to identify any remaining soft areas. The proof roller must be sized to avoid causing bearing capacity failures in the materials that are being proof rolling. Proof rolling is not a replacement for good compaction procedures and inspection. QC/QA personnel need to be present on site to watch the deflections under the roller in order to identify soft areas. Construction equipment, such as loaded scrapers and material delivery trucks, can also be used to help detect soft spots along the highway alignment. Details on the determination of suitability using proof rolling methods were provided in Section 8.3.

Table 8-4. Recommended field compaction equipment for different soils
(*after Rollings and Rollings, 1996*).

Soil	First choice	Second choice	Comment
Rock fill	Vibratory	Pneumatic	—
Plastic soils, CH, MH (A-7, A-5)	Sheepsfoot or pad foot	Pneumatic	Thin lifts usually needed
Low-plasticity soils, CL, ML (A-6, A-4)	Sheepsfoot or pad foot	Pneumatic, vibratory	Moisture control often critical for silty soils
Plastic sands and gravels, GC, SC (A-2-6, A-2-7)	Vibratory, pneumatic	Pad foot	—
Silty sands and gravels, SM, GM (A-3, A-2-4, A-2-5)	Vibratory	Pneumatic, pad foot	Moisture control often critical
Clean sands, SW, SP (A-1-b)	Vibratory	Impact, pneumatic	—
Clean gravels, GW, GP (A-1-a)	Vibratory	Pneumatic, impact, grid	Grid useful for over-sized particles

It is very difficult to achieve satisfactory compaction if the lift is not on a firm foundation. Figure 8-11 shows a typical stress distribution under a rubber-tired pneumatic roller for two different foundations. The first case corresponds to a homogeneous deposit with a constant modulus of elasticity equal to 170 MPa (25,000 psi), which is representative of a good quality granular material. The second case corresponds to a 150 mm (6-in.) thick lift of 170 MPa (25,000 psi) granular material over a subgrade soil having a modulus of 35 MPa (5,000 psi), which is representative of a soft clay having a CBR of around 3 or 4. As is clear from the figure, the stresses induced by the roller in the second case are much lower than in the first. High levels of compaction will be difficult to achieve in the thin lift over the weak subgrade, and the high stresses in the lower soil may produce shear failure and excessive rutting, especially if proof rolling is performed. Thus, it is easy to see the importance of monitoring cut surfaces through proof rolling and measuring compaction of each lift during fill placement.

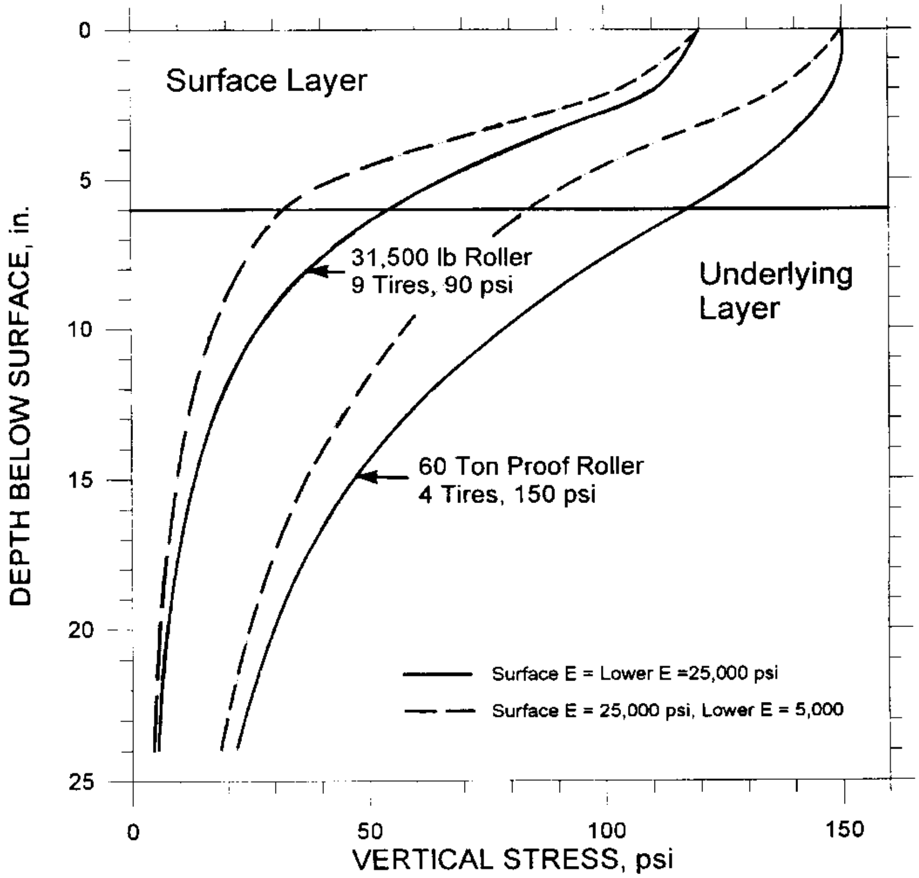


Figure 8-11. Stress distributions under rollers over different foundations (*after Rollings and Rollings, 1996*). (1 ton = 8.9 kPa, 1 psi = 6.9 kPa)

The most common measure of compaction is density. Field moisture and densities can be measured using a variety of standard methods. Field density is correlated to moisture-density relationships measured in the lab (AASHTO Test Procedures T99 and T180). Moisture-density relationships for various soils are discussed in Chapter 7, and the lab tests are covered in Chapter 5. Optimal engineering properties for a given soil type occur near its compaction optimum moisture content (w_{opt} or OMC), as determined by the laboratory test standard. At this state, a soil's void ratio and potential to shrink (if dried) or swell (if inundated with water) is minimized.

In controlling compaction, the appropriate moisture-density laboratory method (*e.g.,* standard or modified Proctor) should be matched to the equipment typically used in the local region. Higher energy equipment should be controlled with compaction tests based on high energy (*i.e.,* modified in lieu of standard Proctor). There is a trend to lower moisture content tolerances with consideration for the higher energy equipment; however, this method could result in lower compactive efforts. The reason for this move is that the high-energy compaction equipment is causing an apparent pumping to occur when the soil is above its optimum moisture. However, this method could ultimately lead to premature failures as the subgrade saturates over time (*e.g.,* loss in stiffness/strength, or potential volume change). It is considered good practice to compact at the optimum moisture content for the material used. If some deviation occurs, it is better to be on the wet side, rather than dry.

Compaction, or mechanical stabilization (*i.e.,* water content adjustments and densification) is the most common and least expensive of all soil improvement techniques. Perhaps the most common problem arising from deficient construction is related to mechanical stabilization. The intent of mechanical stabilization is to maximize the soil strength (and minimize the potential volume change) by the proper adjustment of moisture and the densification at or near the ideal moisture content, as discussed in Section 8.4.4. Without proper quality control and quality assurance (QC/QA) measures, some deficient work may go unnoticed. This is most common in utility trenches and even bridge abutments, where it is difficult to compact because of vertical constraints. This type of problem can be avoided or at least minimized with a thorough plan and execution of the plan as it relates to QC/QA during construction, as reviewed in detail in Section 8.3. This plan should pay particular attention to proper moisture content, proper lift thickness for compaction, and sufficient configuration of the compaction equipment utilized (weight and width are the most critical). Failure to adequately construct and backfill trench lines will most likely result in localized settlement and cracking at the pavement surface.

There are several compaction devices available in modern earthwork, and selection of the proper equipment is dependent on the material intended to be densified. Generally, compaction can be accomplished using pressure, vibration, and/or kneading action. Heavy equipment, as measured by ground pressure, is utilized to accomplish the compaction process. Some heavy equipment have low ground pressures, such as tracked vehicles like bulldozers, and rubber-tire equipment like front end loaders and motor graders. The low ground pressures imparted by these types of vehicles are not effective at compacting soils; however, tracked vehicles do provide some limited compaction of granular, cohesionless materials by means of vibration.

A *smooth drum roller* is perhaps the most common of all compaction devices, capable of applying pressure across the width of its drum. Smooth drum rollers can consist of a single drum (Figure 8-12) or dual drum. Most drum rollers are equipped with oscillary vibrators to increase the energy transmitted to the surface of the layer being compacted. These smooth drum rollers are best suited for granular, relatively non-cohesive soils. Some agencies have used smooth drum rollers to finish subgrades prior to base construction, and have even employed them as a proof rolling instrument.

Figure 8-12. Smooth drum roller (*photo courtesy of Bomag*).

The *sheepsfoot or studded rollers* like the one shown in Figure 8-13 are typically used on cohesive soils. These rollers are very similar to the smooth drum roller, however, many rounded or rectangular protrusions (or feet) are attached to the drum. These protrusions provide for a very high contact pressure in a small zone of soil. By spacing these protrusions apart, very high vertical stresses, as well as horizontal stresses, are achieved, thus creating a kneading action that compacts from the bottom up. During compaction, the roller literally "walks out" of the lift once compaction is achieved. This kneading or shearing action has the ability to produce a soil structure that maximizes a cohesive soil's strength at high density levels. Some sheepsfoot or studded rollers are also equiped with oscillatory vibrators to increase the effectiveness across a broader range of soil.

Pneumatic or *rubber-tire rollers* have also been utilized to compact materials. These compactors are typically used as an alternate for compacting a variety of soil types (see Table 8-4). They are particularly effective for non-cohesive silty soils. Some agencies have used them successfully in embankment placements and have also employed them as a proof rolling instrument. Hauling vehicles (scrapers and loaded dump trucks) have been used for compaction purposes.

The latest compaction equipment are *high-energy impact rollers*, which use shaped (*e.g.,* triangular ellipsoids or hexagonal), as opposed to round drums, as shown in Figure 8-14. The high energy imparted by these systems allows them to achieve compaction at a faster rate and to greater depths. A comparison of different types of compaction equipment based on vertical settlement with number of passes is shown in Figure 8-15, demonstrating the superior effectiveness both in terms of number of passes and influence depth of high-energy equipment.

Most of the research on this equipment has been performed in Europe, and unfortunately the availability is limited at this time in the U.S. The Europeans are also experimenting with hydraulic and pneumatic impact hammers to achieve compaction at greater depths, especially in rubblized fills (Dumas, et al, 2003). This technique uses a 5-tonnes (5.5 ton), 1-m (3.3 ft) drop hydraulic pile hammer to drive a large foot into the ground. This technology eliminates excavation and allows for compaction of shallow layers (or soils with low moisture content) up to 3 m (9 ft) thick. The technique was initially developed by the British and U.S. military for rapid airfield repair.

Figure 8-13. Sheepsfoot roller (*photo courtesy of Bomag*).

Figure 8-14. Impact roller.

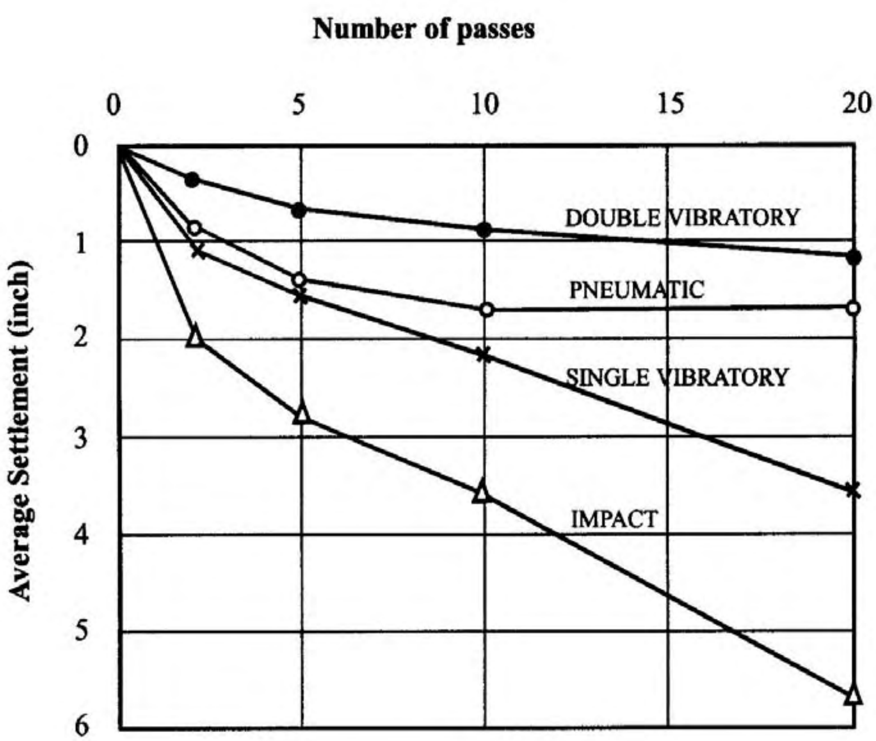


Figure 8-15. Compaction efficiency. (1 in. = 25 mm)

Another significant development in compaction equipment is the use of instruments in the compaction drums to measure the response of soil (*e.g.,* stiffness). The equipment is computerized, allowing for real-time monitoring of foundation response and automated feedback controlling vibration amplitude and frequency, and vehicle speed. While *intelligent compaction equipment* was originally developed for contractors to improve their efficiency in achieving compaction with a minimum number of passes, it has direct and significant application potential for controlling and monitoring compaction effectiveness for pavement performance, as discussed in Section 8.3 on QC/QA.

Other means exist in which to promote deep densification, including dynamic compaction and vibroflotation. These processes are discussed in the FHWA/NHI *Ground Improvement Techniques* reference manual (FHWA NHI-04-001) and, given the right conditions, can be used to densify soils at depths of over 9 meters (30 ft). Each is limited to successes achieved in deep, loose non-cohesive soils, such as sands and gravels.

As previously discussed, with the advent of newer higher energy compaction equipment, agencies should carefully evaluate their current specifications to meet these changing demands.

The final phase of subgrade construction is the confirmation of surficial support prior to placement of the base/subbase layers. One or more of the methods outlined in Section 8.3 should be utilized (*e.g.,* proof rolling, DCP, FWD).

8.4.5 Stabilization

In certain instances, when stabilization is a more economical means of constructing a pavement section with the desired support characteristics, use of chemical admixtures, such as lime, flyash or cement, is common. These mixtures are typically designed in a controlled laboratory environment in order to establish volumetric properties, such as admixture design content, maximum density, moisture content, and strength, as discussed in Chapter 7.

Chemical Stabilization/Modification. In the special case where lime (or other pozzolanic modifier such as cement or flyash) is to be utilized to enhance the load carrying capability of the soil, the additional effort of introducing the modifier to the soil and mixing prior to compaction is required.

The basic construction steps for chemical stabilization of subgrade soils are (1) pozzolan delivery and distribution; (2) mixing; (3) compacting; and (4) curing. Pozzolans can be applied to a soil either dry or as a slurry. In the case of dry lime, the lime may be either in the form of dry hydrated lime, which is very fine-grained and, thus, may pose dust control problems, or dry quicklime, which is granular and much less dusty.

The pozzolanic material specified for stabilization or modification is distributed along the road alignment, either via bags that are spread manually, by pneumatic trucks with spreader bars, or by dump trucks with controlled tailgate openings. Lime slurries can be mixed in a central mixing plant or in various types of portable mixing systems. A typical lime slurry mixture would consist of 0.9 tonne (1 ton) of lime mixed with 1900 liters (500 gal) of water to produce 550 tonnes (600 tons) of slurry with 31% lime solids (Transportation Research Board, 1987).

Adequate mixing of the pozzolanic material with the soil is critical; poor mixing is the leading cause of unsatisfactory stabilization results. Subgrade soils can be mixed on site with the pozzolan by disking, repeated blading, or by traveling rotary or pug-mill mixing equipment.

Figure 8-16. Roadway stabilizer/mixer (*photo courtesy of Bomag*).

Mixing is usually done in thin lifts and often with multiple passes, with the lift thickness and number of passes dependent upon the soil type and the mixing equipment being used. A two-stage mixing process is sometimes used for highly plastic materials; the reduced plasticity and coarser texture that develops during curing for several days after the initial mixing makes the soil more workable for final mixing and compaction.

Compaction of chemically stabilized soil mixtures follows standard procedures. However, with respect to lime stabilization, the addition of lime will generally decrease the maximum density and increase the optimum water content at a given compaction energy, which may cause problems determining the percentage of specified density achieved by the field compaction. Compaction curves of the in-situ lime-soil mixture at the time of compaction may be required to determine the appropriate density values for field compaction control.

Curing at temperatures above 4.4^{o} C (40^{o} F) and with adequate moisture is essential for the pozzolanic reactions underlying the long-term strength gains in lime-stabilized soils. A cure period of 3 – 7 days is typically employed, with adequate moisture maintained either through moist curing (*e.g.,* truck sprinklers) or by applying an asphalt seal over the surface.

Similarly, modified soil (lime, cement and/or flyash) will require special QC/QA considerations, as discussed in Section 8.3. Again, a final evaluation of the stabilized subgrade surface should be made by one or more of the methods described in Section 8.3 (*i.e.,* proof rolling, DCP, FWD).

8.4.6 Base and Subbase Construction

In the case of aggregate base construction, material is hauled to the site and is typically placed directly on grade, spread to a uniform specified thickness, and compacted. Care must be exercised to minimize segregation of aggregate blends. Good practices to prevent or minimize segregation include eliminate the number of transfer points prior to final grade placement (avoidance of intermediate stockpiling), and minimize the amount of spreading and movement once on grade. Asphalt pavement spreaders have been successfully utilized to distribute aggregate base materials on grade. Use of such equipment allows good control of specified thickness and reduces the potential of segregation caused by traditional spreading techniques such as motor-grader or dozer operations. This practice can also positively affect the overall project profile smoothness objectives. Regardless of the method of placement, care should be exercised to avoid the potential to contaminate the aggregate with site soil. Contamination is typically introduced when wet soils adhere to construction traffic tires or tracks, and are "cleaned off" when traversing over newly placed aggregate layers. Again, QC/QA personnel should document observations and test results, indicating conformance or non-conformance with the specification.

Base and subbase layers are typically aggregate materials containing moderate (dense graded) to little (open graded or drainable) fines. Compaction is typically achieved utilizing vibratory smooth drum compactors described previously in Section 8.4.4. Failure to achieve proper compaction may be a result of several factors, either individually or in combination:

- Lack of substrate support (Should have been detected and corrected prior to placement of layer.)
- Improper size of compactor
- Excessive moisture (Perhaps from rainfall. Site surface runoff should be promoted. Excess moisture usually can be dried by blading and allowing excessive moisture time to evaporate.)
- Segregation

Correction in the form of drying and recompacting should work in the majority of cases. If problems persist, removal and replacement may be warranted. If the problem is deeper than the base or subbase layer, subexcavation and replacement or some form of chemical stabilization technique may be required, as discussed in Section 8.4.5.

Chemical Stabilization. Mixing of lime or cement with coarse aggregates for base and subbase layers is often done in a central mixing plant. Although the central mixing plant is required primarily for gradation control, it also enables good control of the lime-aggregate or cement-aggregate proportions and mixing. Again, testing during construction should closely parallel that described for soil stabilization/modification.

8.4.7 Pavement Drainage Systems

For construction of pavement drainage systems, design should acquaint agency construction personnel with the impact of construction on the design results. Care during construction to build the pavement drainage designed section without compromising the effectiveness of design is essential to the pavement's long-term performance. Key performance elements for construction personnel to remember include

- Good pavement starts with a good foundation. A stable platform is required for construction of the permeable base.
- Quality of aggregate and its ability to meet gradation requirements is essential to meeting design performance.
- An awareness that the introduction of fines into the permeable base during construction could result in the premature failure of the pavement.

- Unstabilized drainable base tends to displace under traffic.

In addition to these key elements, construction personnel (contractor and inspector) should be aware of how each construction activity can impact the performance of the pavement drainage system.

Subgrade Preparation As with all road sections, the foundation surfaces are required to be level, somewhat smooth and constructed to required grades. With drainable pavement sections, constructing and maintaining the required subsurface grades until pavement construction is essential in maintaining positive pavement drainage. Grades that are too flat, local depressions resulting from soft areas, and/or depressions from equipment trafficking can lead to ponding of water below the pavement structure and subsequent loss of foundation support.

Separation Layers For granular separation layers, the gradation of materials must be carefully checked against design requirements. Material that is more open than specification requirements may allow migration of fines and contamination of the permeable base. Good compaction of the separation layer is essential to the placement of the permeable base. The subbase should be observed for rutting during compaction and subsequent trafficking. Subbase surface rutting may be an indication of subgrade rutting and requires immediate attention (*e.g.,* by reducing equipment loads or increasing the lift thickness). "A separator is not a substitute for proper subgrade preparation" (FHWA-SA-92-008).

For construction of geotextile separation layers, material and certification should be checked against design specification requirements to make sure the proper materials have been received and used. The smooth subgrade surface is desirable. It is recommended that sharp rock protrusions or loose rocks (usually greater than 20 mm (¾ in.) in size) be removed to avoid damage to the geotextile, unless such conditions have been anticipated and heavy-weight (greater than 250 g/m^2 (7.4 oz/yd^2)) geotextiles have been specified.

Edgedrains Proper grading is essential for edgedrains to be effective. Undulating drain lines are not acceptable, as water will accumulate in depressed areas. Good practice dictates that drains must be properly connected to the permeable base and to outlets. Outlets are required to be set at the proper grades and ditch lines graded according to drainage requirements. Drain lines are to be carefully marked and care maintained throughout construction to avoid crushing the pipe with construction equipment (*e.g.,* concrete trucks and other heavy vehicles/equipment are not to be allowed to travel over drain lines). Drains are sometimes constructed after pavement construction to avoid this problem. In this case, temporary drainage is required for the permeable base to prevent a bathtub effect from water trapped in the porous base.

As discussed in Section 7-2, the filter (geotextile or aggregate) has to be carefully placed at the design location around all sides of the backfill, not in contact with the permeable base.

The edgedrains are required to be backfilled with material at least as permeable as the permeable base. Most states use a graded aggregate, while some states use free-draining sand. In either case, the drainage backfill should be placed below the invert of the pipe, and compacted to better support the pipe, reduce the risk of crushing the pipe, and to prevent subsequent subsidence that could affect the road. As with the trench line, the pipe must be placed at the proper grade on a level surface. Drainage backfill is placed to the final elevation and protected from fouling until the pavement section is complete. Maintaining an open drainage aggregate is critical during the remaining construction period. A shovel full of fines could clog the drain. Construction traffic should not be allowed to traverse over the drain line. The drain line could be covered with a geotextile to help prevent fouling during construction. Also, outlets must properly drain during this phase to provide temporary drainage during construction. Ditch lines should be continuously checked and maintained, as erosion sediments could back up and foul essential features. Headwalls for outlets should be installed and outlets marked so they will not be disturbed by subsequent construction.

The edgedrain system should be inspected and tested for proper operation toward the end of construction, before final acceptance. An acceptance criteria based on performance parameters must be established, otherwise signs of poor construction practices will most likely not be identified until major structural damage is done and the pavement life has been shortened. Inspection techniques can consist of simply pouring water on the drainage layer in an upstream section of the drain and measuring the outflow against the anticipated rate. The most effective method for post-construction evaluation is video equipment (*e.g.,* Iowa borescope and other mini-cameras). Several states do not accept edgedrains until video inspection indicates that they have not been damaged during construction. The design of the edgedrain system should have included pipe access installed at the "upstream" end of the drain line to gain access for camera inspection, effectiveness testing, and subsequent maintenance flushing activities.

Drainable Base Materials Unstabilized permeable base requires close control of the material gradation and attention to activities that might cause segregation. An asphalt spreader box is usually required to reduce segregation. Unstabilized base tends to weave and rut under traffic.

Asphalt-stabilized permeable base usually contains AASHTO No. 67 or No. 57 crushed aggregate plus 2 – 2.5% asphalt by weight. Higher asphalt cement percentages may be required when a less-open gradation is used. Some states prohibit the use of bank run gravel aggregate because of the rounded faces. Stabilized aggregate should be placed at 90 – 120° C (200 – 250° F) but not rolled until it is below 65° C (150° F). Vibratory rollers are usually not allowed, and the number of roller passes is usually between 1 and 3 (FHWA-SA-92-008).

Cement-stabilized permeable base usually contains 2 to 3 bags for No. 67 and No. 57 crushed aggregate. As with asphalt-stabilized base, higher amounts may be required for less-open graded aggregate. Cement-stabilized base could be cured similar to pavement. Test strips are recommended to determine appropriate curing and compaction methods (FHWA-SA-92-008).

Care is required to protect the permeable base from fines contamination (*e.g.,* from dirty construction equipment, adjacent backfilling operations, erosion sediments, etc.). While the drainable base can generally support light construction loads, it should not be used as a haul road. Equipment that would cause rutting (*e.g.,* concrete and loaded dump trucks), dirty equipment, or equipment transporting fines should not be allowed to traverse over the permeable base. Good practice dictates that traffic be restricted to low speeds with minimal turning allowed. Traffic should not be allowed until complete drainage of the base and subbase has been confirmed.

Based on a survey of state agencies (Christopher and McGuffy, 1997), good construction of subsurface drainage systems appears to depend on a number of factors:

- The contractor (and inspector) should be knowledgeable in drain installation principles and practices.
- Someone with knowledge of drainable pavements must be on site at startup.
- Water needs a continuous, unobstructed path to drain, both during and after construction.
- A positive slope is required.
- Any discontinuity in flow path can destroy the system's effectiveness.
- The pavement (or shoulder) is supported by the system; therefore, compaction is essential.
- Construction activities for other work in the area can destroy good drainage installations.

8.5 PERFORMANCE MONITORING

8.5.1 Pavement Management Systems

Pavement management systems have been utilized as tools to document and track pavement performance. These systems typically rely on the assessment of the pavement wearing surface, in the form of distress surveys performed at periodic intervals, in order to not only illustrate how the pavement is performing, but to predict how the pavement may perform into the future. Through the use of these tools, agencies have been able to detect performance problems early, and correct the problems with routine maintenance during the pavement life-cycle. These tools assist agencies to best manage maintenance and capital budgets across their broad network of pavements, and can be utilized efficiently at the project level to optimize pavement performance for individual construction projects. These tools become very important at the project level when considering performance risk, particularly with extended performance periods.

A major disadvantage with the conventional distress survey input for a pavement management system, particularly with respect to pavement layers associated with unbound materials, is that problems are not detected until failure occurs. Problems caused by moisture intrusion into the subgrade and unbound base/subbase layers weaken the pavement system. If gone undetected, a pavement's life can be dramatically shortened. In order to circumvent this problem, agencies and particularly design-build teams, have seen the benefit of augmenting a solid pavement management system (distress survey) with structural surveys (NDT using one of the many geophysical testing techniques previously documented in Chapter 4, and described in further detail in the following sections).

8.5.2 Geophysical

Geophysical measurements detect differences or anomalies in material properties. However, these properties require interpretation as conditions relevant to pavement performance. As discussed in Chapter 4, geophysical testing can be used to locate voids beneath pavement sections for both construction and long-term performance monitoring. Periodic monitoring of a region with known problems such as solution caves or other karstic features can be a significant asset in evaluating the effectiveness of grouting programs to solve problems during construction and evaluate any long-term developments that could lead to future problems. The following two case histories provide a demonstration of effective use of geophysics in both short- and long-term monitoring programs.

As indicated in Chapter 4, the Finnish government performs resistivity testing on subgrades along with other in-situ and geophysical tests to develop a complete map of the subgrade system, including moisture and corresponding settlement and frost heave profiles. These anticipated profiles are then used to define the performance requirements for roadway warranties. The allowable settlement for a 30-year service period and a 5-year warranty period is calculated based on this well-documented and detailed site investigation (Tolla, 2002).

Widening and realignment of State Route 69 traverses an area of Tertiary-age travertine bedrock near Mayer, Arizona. During the design phase of the project, subsurface exploration encountered small voids within the right-of-way. A moderate-sized cave structure in the area was mapped by local speleologists. Arizona Department of Transportation (ADOT) was concerned that cave structures of unknown size might be found within a few feet of the new roadway subgrade. As a result, highway construction specifications contained special provisions requiring geophysical surveys to identify cave structures that could adversely affect the roadway and expose the traveling public to possible subgrade failure hazards. ADOT's concern was realized during construction when a D-9 Caterpillar tractor broke through a cave roof and dropped about 1.8 m (6 ft) into the void. A geophysical survey conducted of the cave-affected alignment identified 130 cave-type anomalies, and recommendations were provided to ADOT and the contractor to remediate the cave-affected highway section. Survey monuments were established for monitoring roadway performance and potential subgrade settlement (Euge et al., 1998).

8.5.3 Falling Weight Deflectometer

Much research has been conducted by FHWA in the past decade, particularly as part of the Long-Term Pavement Performance (LTPP) study. Although typically utilized as a tool to measure structural capacity of a pavement system for the primary purpose of designing strengthening and overlay thickness requirements, the FWD can be utilized to monitor the subgrade performance, as well as base/subbase performance. This type of program can be established by first measuring the deflection profile of a newly completed or rehabilitated pavement section at numerous discrete points (baseline data). These measurements (particularly deflections away from the loaded plate) can provide useful information about the deeper layers in the pavement system. Measurements made at annual or seasonal periods and compared with baseline data may indicate when potential problems exist. A loss in stiffness in a deep subgrade, or intermediate base or subbase layer, may indicate a poor drainage condition exists, one which can be readily corrected prior to premature pavement system failures by either constructing underdrains, maintaining existing underdrains, or altering the site hydrology in a way that better promotes site drainage.

8.5.4 Drainage Inspection (*e.g.,* video logging)

Performance monitoring of drainage systems is essential for both acceptance of the constructed facility and for maintaining a preventive maintenance program (NCHRP Synthesis 285). Probably the most significant development in edgedrain inspection has been the use of small diameter, optical tube video cameras with closed circuit video systems. Video cameras allow the inside of the edgedrain system to be logged, and expose the weaknesses in construction and inspection procedures. Iowa was one of the first states to effectively use video inspection *(Steffes et al., 1991)*. Random inspection of drains with video cameras has exposed many problems including

- rodent nests in the drain,
- varied sag from main line to outlet,
- polyethylene tubing and connector failures,
- break from stretch or puncture, and
- geocomposite drain J-buckling.

As was discussed in Chapter 4, significant effort to evaluate the use of video cameras as an inspection tool and demonstrate the technology was undertaken by the Federal Highway Administration. In evaluation of 269 outlet pipes that were inspected, 35% of the laterals could not be inspected because they were crushed or clogged, and the condition of the mainline could not be investigated. Of the mainlines that were evaluated, 17% were blocked or clogged. These findings clearly indicated that there were serious inadequacies in the edgedrain design, construction, and maintenance practice. The study also showed that the video inspection of edgedrains was a viable tool for determining the existing condition of edgedrains and had a definite role in providing construction quality assurance.

The Federal Highway Administration program to promote this technology appears to have had a significant impact. Over 17 states reported to have used a video camera. Many agencies own their own video camera, with a cost for the system ranging from \$13,000 – \$40,000. Some agencies retain consultants to perform video inspections. Video cameras have proven to be a valuable tool for many of the agencies in identifying problems and exposing weaknesses in construction and inspection procedures. Many states currently do or will shortly require video inspection for construction acceptance. Several agencies have reported that they have improved from an edgedrain failure rate of up to 40% to a failure rate of less than 5% by improving their QC/QA program, including the use of video cameras. Several agencies have incorporated their video camera into their preventive maintenance program, with periodic monitoring during routine inspection.

8.5.5 Instrumented Geosynthetics

Geosynthetics provide a convenient delivery system for performance monitoring instruments. Instrumentation, including strain gauges for deformation and stress measurements, pore pressure transducers to monitor soil suction, dielectric sensors to monitor moisture change, and thermistors to measure temperature change, can all be installed in the factory, delivered to the site, and hooked directly into a remote data acquisition system with telecommunications (no wires). Geosynthetics are currently available in Europe with an array of strain gages embedded in the product for monitoring subsidence (*e.g.,* from karst conditions and abandoned mines). This allows performance monitoring with practically no disruption to construction. Care is still required to avoid damage to the instruments during placement of the section and the initial fill over that section.

8.6 POST-CONSTRUCTION ISSUES AND SPECIAL CASES

The installation of structural features (*e.g.,* storm water lines and manholes, culverts, roadway drainage lines, etc.) adjacent to or beneath pavements can also lead to problems during or following construction. Proper installation of such structures is critical and close inspection during construction is critical. For example, a precast concrete pipe is installed as a storm drain. Each segment of the pipe is grouted, and the pipe is grouted into a junction box that also serves as a storm drain surface inlet (surface grate). The pipe is located on a 100-mm (4-in.) sand bedding at the bottom of a trench excavation. Following installation of the pipe, the trench is backfilled adequately, and the pavement is constructed. Imagine though, that one of the pipe joints was not adequately grouted, or post-construction settlement occurred (*e.g.,* due to inadequate embankment or bedding compaction) causing differential movement such that one of the joints separates. Because of the amount of storm water carried by the pipe during the most intense rainfall, the turbulent flow of water has begun to pipe soil backfill from around the pipe, and has swept it to the junction box and further down stream. The progression of this piping and erosion will eventually lead to pavement subsidence.

This type of pavement failure, subsidence of underlying strata, can be manifested by a mechanism as described above or by a similar mechanism – water movement through voids, piping or eroding fines over time to cause larger voids that eventually collapse. These failures, described as *sinkholes*, generally are catastrophic in nature, and costly to repair (construction and delay costs). A key element in the installation of piping systems is proper compaction beneath and around the pipe. Granular fill should always be used to form a haunch below the pipe for support. Some state agencies are using flowable fill or controlled low-strength material (CLSM) as an alternative to compacted granular fill (NCHRP Project 24-12). Without this support feature, the weight above the pipe will cause it to deform laterally, creating settlement above the pipe and often pipe collapse. Even if a sinkhole does not appear, leaks of any water bearing utility will inundate the adjacent pavement layers reducing their support capacity. Several agencies have used CLSM around pipes in the pavement section.

Pavement problems also occur when improper fill is used in the embankment beneath the pavement system. Placement of tree trunks, large branches and wood pieces in embankment fill must not be allowed. Over time, these organic materials decay, causing localized settlement and, eventually, voids in the soil. Again, water entering these voids can lead to collapse and substantial subsidence of the pavement section. Likewise, placement of large stones and boulders in fills creates voids in the mass, either unfilled due to bridging of soil over the large particles or filled with finer material that cannot be compacted with conventional equipment. Soil above these materials can pipe into the void space creating substantial subsidence in the pavement section. These issues can be mitigated with a well-crafted specification that will not allow these types of materials to be used, and full execution of the project QC/QA plan. (*e.g.,* Uniformity Coefficient, $C_u = D_{60}/D_{10} > 15$, <u>and</u> Coefficient of Curvature, $C_c = D_{30}^2/D_{10}D_{60} > 5$).

Special cases may require large stone (*e.g.,* blast rock or surge stone) to be used as fill. If such material must be used as fill, then select graded granular material and/or geotextiles should be placed above and below the large stone to form separator layers. For the use of a granular separation layer, the upper layer should be well compacted to choke off the voids in the stone. If possible, this layer should be flooded with a hose to confirm its compatibility. The gradation of the granular material must be such that it will not move into the void space and must also meet the filter criteria for the finer-grained fill material in the embankment. These conditions are met if the following gradation criteria are satisfied:

- D_{85} graded granular blanket $\geq 0.2\ D_{50}$ large stone fill
- D_{85} graded granular blanket $\leq 5\ D_{85}$ embankment fill
- D_{15} graded granular blanket $\geq 5\ D_{15}$ embankment fill

An alternative is to use high-survivability geotextiles (AASHTO M288 Class 1) placed immediately above and below the large stone layers to act as separators and prevent soil from moving into the void spaces in these materials.

Transitions between cut zones and fill zones can also create problems, particularly related to insufficient removal of weak organic material (clearing and grubbing), as well as neglect of subsurface water movements.

A specific transition also occurs at bridge approaches. These problems are typically related to inadequate compaction, typically a result of improper compaction equipment mobilized to the site, lack of supervision and care (*e.g.,* lift placement greater than compaction equipment can properly densify).

Many problems arise only after construction is completed and some amount of service life has been consumed. Detection of problems attributed to the geotechnical aspects of the pavement can be identified by interpretation of distresses observed. Examples of problems associated with flexible and rigid pavements are highlighted in Tables 8-5 and 8-6, respectively.

Pavement design and construction is an ongoing voyage. The industry has encountered numerous untold problems, and has found logical and economical solutions to mitigate these problems in the design stage, the construction stage, and the performance stage. Local agencies have developed their own strategies to solve anticipated problem and deal with unanticipated problems. This chapter was intended to summarize and discuss the majority of these issues.

Table 8-5. Geotechnical related post-construction problems in flexible pavements.

Problem/Distress Observed	Probable Cause(s)	Corrective Action
Longitudinal crack in wheel path (fatigue)	1. weak subgrade 2. insufficient pavement thickness	overlay
Surface rut in wheel path	1. over-stressed subgrade 2. post-construction densification of asphalt layer(s)	- leveling course in ruts and overlay - plane and overlay (*i.e.,* mill & fill)
Staining in surface cracks – color of stains consistent with local soil	drainage problems, wet subgrade – fines contamination in base, if present	- reconstruction - thick overlay (extend life somewhat, but mask the problem)
Intermittent depressions, subsidence	1. erratic compaction control 2. buried organic matter 3. piping in subsurface voids (*e.g.,* around utilities)	localized demolition and patching
Edge cracking	frost susceptibility and drainage issues	Construct wider shoulder; use materials that are not frost susceptible above the depth of frost penetration.

Table 8-6. Geotechnical related post-construction problems in rigid pavements.

Problem/Distress Observed	Probable Cause(s)	Corrective Action
Staining in surface cracks and/or joints – color of stains consistent with local soil	drainage problems, wet subgrade – fines contamination in base, if present, pumping fines	- reconstruction - thick overlay (extend life somewhat, but mask the problem)
Corner break	pumping of fines resulting in void or loss of support beneath slab	localized demolition and patching or reconstruction depending on extent of problem (mud-jacking and under sealing may be a good option if voids are detected prior to breaks)
Faulting at joints, subsidence of utility patches	1. erratic compaction control 2. buried organic matter 3. piping in subsurface voids (*e.g.,* around utilities) 4. pumping of fines resulting in void or loss of support beneath slab	localized demolition and patching or reconstruction depending on extent of problem (mud-jacking and under sealing may be a good option if voids are detected prior to faults, diamond grinding may be required for smoothness)

Example 8.1. A class exercise will be constructed around problems that are common to the agency. Each team will be given a specific pavement subgrade scenario and asked to identify the most appropriate soil improvement method(s) and describe the reasons for their selection. Other teams will critique the selection(s). This exercise would then be followed by slides summarizing the advantages and disadvantages of all of the soil improvement methods.

8.7 REFERENCES

AASHTO (2004). *AASHTO Standard Specifications for Transportation Materials and Methods of Sampling and Testing (24th Ed.)*, American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO (1993). *AASHTO Guide for Design of Pavement Structures*, American Association of State Highway and Transportation Officials, Washington, D.C.

Boudreau, R.L. (2003). *Resilient Modulus – Pavement Subgrade Design Value*, ASTM STP 1437 - Resilient Modulus Testing for Pavement Components, G.N. Durham, A.W. Marr, and W.L. De Groff, Eds., American Society for Testing and Materials, West Conshohocken, PA.

Chamberlin, W.P. (1995). *Performance-Related Specifications for Highway Construction and Rehabilitation*, National Cooperative Highway Research Program Synthesis 212.

Christopher, B.R. and McGuffey, V.C. (1997). Pavement Subsurface Drainage Systems, National Cooperative Highway Research Program, Synthesis of Highway Practice 239, Transportation Research Board, National Academy Press, Washington, D.C.

Daleiden, J. (1998) *Video Inspection of Highway Edgedrain Systems*, Final Report, Brent Rauhut Engineering.

Dumas, C., Mansukhani, S., Porbaha, A., Short, R.D., Cannon, R.R., McLain, K.W., Putcha, S., Macnab, A., Lwin, M.M., Pelnik III, T.W., Brown, D.A., Christopher, B.R. (2003) *Innovative Technology For Accelerated Construction Of Bridge And Embankment Foundations In* Europe, Prepared by American Trade Initiatives for Federal Highway Administration, http://isddc.dot.gov/OLPFiles/FHWA/010940.pdf.

Euge, K M; Hennon, K; Williams, J C; Priznar, N M (1998). "Ground Penetrating Radar: an Effective Tool for Locating Dry Caves along a Portion of State Route 69 Near Mayer, Yavapai County, Arizona," *Proceedings and Field Trip Guide of the 49th Highway Geology Symposium*. Highway Geology Symposium.

FHWA NHI-04-001 (2004) *Ground Improvement Methods*, Participant Notebook, NHI Course 132034, authors: Elias, V., Welsh, J., Warren, J., Lukas, R., Collin, J.G., and Berg, R.R., National Highway Institute, Federal Highway Administration, Washington, D.C., 1022 pp.

FHWA NHI-00-045 (2000). Soils and Foundations Workshop, NHI Course No. 132012, Federal Highway Administration, Washington, D.C.

FHWA-SA-92-008 (1992). *Demonstration Project 87: Drainable Pavement Systems, Participant Notebook*, Federal Highway Administration, Washington, D.C.

Hancher, D.E. (1994). *Use of Warranties in Road Construction*, National Cooperative Highway Research Program Synthesis 195.

Illinois DOT (1982) *Subgrade Stability Manual*, Illinois Department of Transportation.

Joint Departments of the Army and Air Force (1994). *Soil Stabilization for Pavements*, TM 5-822-14/AFMAN 32-8010.

Nazarian, S., Yuan, D., Tandon, V. and Arellano, M. (2002). *Quality Management of Flexible Pavement Layers with Seismic Methods*, Texas DOT Report No. TX-02 1735-3F.

Ohio Department of Transportation (2002). *Subgrade Construction and Stabilization*, Office of Construction.

Rollings, M.P., and R.S. Rollings (1996). *Geotechnical Materials in Construction*, McGraw-Hill, NY.

Steffes, R F; Marks, V J; Dirks, K L (1991) “Video Evaluation Of Highway Drainage Systems,” *Transportation Research Record No. 1329*, Transportation Research Board, Washington, D.C.

Tolla, P. (2002). Presentation to the FHWA Scan Team on “Innovative Technology for Accelerated Construction of Bridge and Embankment Foundations.”

Von Quintus, H.L., Rao, C., Stubstad, R.N., Prowell, B., Maghsoodloo, S., Maser, K.R., Nazarian, S. and Minchen, R.E. (2004). *Nondestructive Testing Technology for Quality Control and Acceptance of Flexible Pavement Construction*, Draft Interim Report, National Cooperative Highway Research Program, NCHRP Project 10-65.

Wisconsin Department of Transportation (2002). *Comprehensive Subgrade Deflection Acceptance Criteria, Executive Summary*.

APPENDIX A. TERMINOLOGY

The following provides a definition of the pavement components, along with other terms common to the geotechnical aspects of pavements as contained in this manual. (Definitions were taken from NCHRP 1-37A, where available).   The terms and definitions are organized under five general headings:

- Primary Pavement Components
- Geotechnical Pavement Components
- Non-Geotechnical Components
- Design Terminology
- Pavement Distress and Failure Terminology

Primary Pavement Components

subgrade - The top surface of a roadbed upon which the pavement structure and shoulders are constructed.

subbase - The layer or layers of specified or selected materials of designed thickness placed on a subgrade to support a base course. Note that the layer directly below the PCC slab is now called a base layer, not a subbase layer.

base - The layer or layers of specified or select material of designed thickness placed on a subbase or subgrade to support a surface course. The layer directly beneath the PCC slab is called the base layer.

surface course - One or more layers of a pavement structure designed to accommodate the traffic load, the top layer of which resists skidding, traffic abrasion, and the disintegrating effects of climate. The top layer of flexible pavements is sometimes called the "wearing" course.

Geotechnical Pavement Components

The **geotechnical components** of a pavement system as covered in this manual include unbound granular base, unbound granular subbase, the subgrade or roadbed, aggregate and geosynthetics used in drainage systems, graded granular aggregate and geosynthetic used as separation and filtration layers, and the roadway embankment foundation. Terms related to these components are defined as follows.

aggregate base (AB) - A base course consisting of compacted mineral aggregates. Also, granular base (GB), unbound granular base.

aggregate subbase (ASB) - A subbase course consisting of compacted mineral aggregates. Also, granular subbase, unbound granular subbase.

asphalt-treated permeable base (ATPB) - A base containing a small percentage of asphalt cement to enhance stability.

asphalt-treated permeable base (ATPB) - A permeable base containing a small percentage of asphalt cement to enhance stability. Also, asphalt-treated open-graded base (ATOGB), asphalt-treated base-permeable (ATB-Perm).

cement-treated base (CTB) - A base course consisting of mineral aggregates blended in place or through a pugmill with a small percentage of Portland cement to provide cementitious properties and strengthening. Also, aggregate cement, cement-stabilized graded aggregate (CSGA), cement-stabilized base (CSB).

cement-treated permeable base (CTPB) - An open-graded aggregate base treated with Portland cement to provide enhanced base strength and reduce erosion potential.

crushed stone base - A base course of designed thickness and constructed of graded and mechanically crushed mineral aggregate compacted above a subbase course or subgrade. Also, aggregate base (AB), graded aggregate base (GAB), and crushed aggregate (CA).

crushed stone subbase - A subbase course of designed thickness and constructed of graded and mechanically crushed mineral aggregate compacted above a subgrade.

dense-graded aggregate (DGA) - A mechanically crushed, well graded aggregate having a particle size distribution such that when it is compacted, the resulting voids between the aggregate particles, expressed as a percentage of the total space occupied by the material, are relatively small.

drainable granular subbase - A subbase constructed of compacted and crushed open-graded aggregate.

geogrid (GG) - a geosynthetic formed by a regular network of tensile elements with apertures of sufficient size to interlock with surrounding fill material, used primarily as reinforcement of base and subbase layers and in stabilization of soft subgrade layers. Also used in overlays for asphalt reinforcement.

geosynthetic - a planar product manufactured from a polymeric material used with soil, rock, earth, or other geotechnical-related material as an integral part of a civil engineering project, structure, or system.

geotextile (GT) - a permeable geosynthetic made of textile materials, used as a separator between base, subbase and subgrade layers, used as filters in drainage features, and used in stabilization of soft subgrade layers. Also used in asphalt overlays as a membrane absorption and/or waterproofing layer.

gravel - Coarse aggregate resulting from natural disintegration and abrasion of rock or processing of weakly bound conglomerate. In geotechnical engineering, the particles of rock that range in size from 76.2 mm (3-in. U.S. sieve) to 4.75 mm (No. 4 U.S. sieve). To be classified as a gravel in the Unified Classification System (UCS), at least 50% of the material must be in this range. (Identification and classification of soils is covered in Chapter 5.)

gravel base - An unbound base course constructed of compacted gravel. May or may not be graded and/or crushed.

gravel subbase - An unbound subbase course constructed of compacted gravel. May or may not be graded and/or crushed.

gravel subgrade - A subgrade where a natural gravel has been used as the roadbed surface or where the native soil has been blended with a gravel additive (a.k.a. gravel-treated subgrade for the second case).

lime-treated subgrade - A prepared and mechanically compacted mixture of hydrated lime, water, and soil supporting the pavement system.

lime-flyash base (LFB or LFA) - A blend of mineral aggregate, lime, flyash, and water, combined in proper proportions and producing a dense mass when compacted.

modified or treated base - The addition of cement or asphalt (typically less than 5%) to unbound base with the primary purpose of improving the stability for construction (*i.e.,* no improvement anticipated for stiffness or structural support).

open-graded aggregate base (OGAB) - A crushed mineral aggregate base having a particle size distribution such that when compacted the interstices will provide enhanced drainage properties. Also, granular drainable layer, untreated permeable base (UPB).

permeable base (PB) - A base course constructed of treated or untreated open-graded aggregate. Also, free-draining base.

prefabricated geocomposite edge drain (PGED) - An edgedrain consisting of an extruded plastic drainage core covered with a geotextile filter (also known as panel drains or fin drains).

roadbed - The graded portion of a highway between top and side slopes, prepared as a foundation for the pavement structure and shoulder.

roadbed material - The material below the subgrade in cuts and in embankment foundations, extending to such depth as affects the support of the pavement structure.

soil aggregate - Natural or prepared mixtures consisting predominantly of stone, gravel, or sand that contain a significant amount of minus 75-μm (No. 200) silt-clay material.

soil cement - A mechanically compacted mixture of soil, Portland cement, and water, used as a layer in a pavement system to reinforce and protect the subgrade or subbase. Also, cement-treated subgrade (CTS).

stabilized granular base - A base course with an unspecified stabilizing material, usually asphalt cement or Portland cement.

stabilized permeable base - A permeable base with an unspecified stabilizing material, usually asphalt cement or Portland cement. Also, bound drainable base.

subgrade - the top surface of a roadbed upon which the pavement structure and shoulders are constructed with the purpose of providing a platform for construction of the pavement and to support the pavement without undue deflection that would impact the pavements performance (NCHRP 1-37A). In this manual, the natural and/or prepared soil materials beneath the pavement structure that deform under pavement loading or otherwise have an influence on the support of the pavement (a.k.a. roadbed, pavement foundation).

Unbound base/subbase - compacted mineral aggregate layer that may be either untreated or treated, but has not been modified sufficiently to provide an increase in stiffness or strength for design.

Non-Geotechnical Components

As indicated in Section 1.1, the **non-geotechnical components** are the surficial pavement layers, including asphaltic concrete, Portland cement concrete, and bound aggregate layers. Terms related to these components are defined as follows:

asphalt concrete (AC) - A controlled mixture of asphalt cement and graded aggregate compacted to a dense mass. Also, hot-mixed asphalt (HMA), hot-mixed asphalt concrete (HMAC), bituminous concrete (BC), plant mix (PM).

asphalt concrete base (ACB) - Asphalt concrete used as a base course. Also, asphalt base course (ABC), asphalt-stabilized base, hot-mixed (ASB-HM), asphalt-treated base (ATB), bituminous aggregate base, bituminous concrete base (BCB), bituminous base (BB), hot-mixed asphalt base (HMAB).

asphalt concrete pavement (ACP) - A pavement structure placed above a subgrade or improved subgrade and consisting of one or more courses of asphalt concrete or a combination of asphalt concrete and stabilized or unstabilized aggregate courses.

asphalt concrete surface (ACS) - Asphalt concrete used as a surface course. Also, dense-graded asphalt concrete (DGAC).

continuously reinforced concrete pavement (CRCP) - Portland cement concrete pavement with no transverse joints and containing longitudinal steel in an amount designed to ensure holding shrinkage cracks tightly closed. Joints exist only at construction joints and on-grade structures.

flexible pavement - A pavement structure that maintains intimate contact with and distributes loads to the subgrade and depends on aggregate interlock, particle friction, and cohesion for stability.

jointed plain concrete pavement (JPCP) - Jointed Portland cement concrete pavement containing no distributed steel to control random cracking; may or may not contain joint load transfer devices.

jointed reinforced concrete pavement (JRCP) - Jointed Portland cement concrete paving containing distributed steel reinforcement to control random cracking and usually containing joint load transfer devices.

lean concrete base (LCB) - A base course constructed of mineral aggregates plant mixed with a sufficient quantity of Portland cement to provide a strong platform for additional pavement layers and placed with a paver.

plain concrete - PCC without reinforcing steel.

Portland cement concrete (PCC) - A composite material consisting of a Portland or hydraulic cement binding medium and embedded particles or fragments of aggregate.

rigid pavement - A pavement structure that distributes loads to the subgrade, having as one course a Portland cement concrete slab of relatively high-bending resistance.

Design Terminology

In the context of current design practice, pavement designers and geotechnical specialists must communicate using **design terms** with consistent definitions. Terms related to design as used in this manual include

analysis period - (a.k.a. performance period) The time period used for comparing design alternatives. An analysis period may contain several maintenance and rehabilitation activities during the life cycle of the pavement being evaluated.

average annual daily traffic (AADT) - The estimate of typical traffic on a road segment for all days of the week over the period of a year.

average annual daily truck traffic (AADTT) - The estimate of typical truck traffic on a road segment for all days of the week over the period of a year.

axle load - The sum of all tire loads on an axle.

axle load spectrum - The full spectrum (distribution) of single, dual, triple, and other axle loads applied to a pavement structure by a given traffic stream.

bound base - The addition of a sufficient amount of cement or asphalt to change the long term stiffness and structural characteristics of unbound base to that of lean concrete.

design life - The length of time for which a pavement structure is being designed, including the time from construction until major programmed rehabilitation.

elastic layer theory - A mathematical process wherein the layers of a pavement structure are all assumed to behave elastically.

equivalent single axle load (ESAL) - A numerical factor that expresses the relationship of a given axle load to another axle load in terms of the relative effects of the two loads on the serviceability of a pavement structure. Often expressed in terms of 18,000-pound (80 kN) single axle loads.

finite element analysis - The finite element method is one wherein rigorous mathematical solution, often employing complex differential equations, of an engineering problem is approximated algebraically. The geometry of the problem is described by discrete elements of finite dimensions that are analyzed through the application of engineering mechanics principles. Results of the finite element analyses are aggregated to approximate the exact mathematical solution.

international roughness index (IRI) - A pavement roughness index computed from a longitudinal profile measurement using a quarter-car simulation at a simulation speed of 50 mph (80 km/h).

life-cycle cost analysis (LCCA) - An economic assessment of an item, area, system, or facility and competing design alternatives considering all significant costs of ownership over the economic life (which encompasses several analysis periods), expressed in equivalent dollars.

longitudinal profile - The perpendicular deviations of the pavement surface from an established reference parallel to the lane direction, usually measured in the wheel tracks.

mechanistic-empirical (M-E) - A design philosophy or approach wherein classical mechanics of solids is used in conjunction with empirically derived relationships to accomplish the design objectives.

pavement performance - Measure of accumulated service provided by a pavement (*e.g.,* the adequacy with which it fulfills it purpose). Often referred to as the record of pavement condition or serviceability over time or with accumulated traffic.

performance period - The period of time that an initially constructed or rehabilitated pavement structure will last (perform) before reaching its terminal condition when rehabilitation is performed. This is also referred to as the design period.

present serviceability index (PSI) - An index derived by formula for estimating the serviceability rating from measurements of physical features of the pavement.

present serviceability rating (PSR) - A mean rating of the serviceability of a pavement (traveled surface) established by a panel rating under controlled conditions. The usual scale for highways is 0 to 5, with 5 being excellent.

reliability - The probability that a given pavement design will last for the anticipated design performance period.

rideability - A subjective judgment of the comparative discomfort induced by traveling over a specific section of highway pavement in a vehicle.

serviceability - The ability at time of observation of a pavement to serve traffic (autos and trucks) that uses the facility.

single axle load - The total load transmitted by all wheels whose centers may be included between two parallel transverse vertical planes 1 m (40 in.) apart, extending across the full width of the vehicle.

tandem axle load - The total load transmitted to the pavement by two consecutive axles whose centers may be included between parallel vertical planes.

traffic growth factor - A factor used to describe the annual growth rate of traffic volume on a roadway.

transverse profile - The vertical deviations of the pavement surface from a horizontal reference perpendicular to the lane direction.

user costs - Those costs realized by the users of a facility. In a life cycle cost analysis, user costs could take the form of delay costs or of changes in vehicle operating costs associated with various alternatives.

weigh-in-motion (WIM) - The process of estimating a moving vehicle's gross weight and the portion of that weight that is carried by each wheel, axle, or axle group, or combination thereof, by measurement and analysis of dynamic forces.

wheel load - The sum of the tire loads on all tires included in the wheel assembly comprising a half axle.

zero-stress temperature - temperature (after placement and during curing) at which the concrete layer exhibits zero thermal stress (at temperatures less than this, concrete exhibits tensile stress).

Pavement Distress and Failure Terminology

Distress refers to conditions that reduce serviceability or indicate structural deterioration. Failure is a relative term. In the context of this manual, failure denotes a pavement section that experiences excessive rutting or cracking that is greater than anticipated during the performance period or that a portion of the pavement is structurally impaired at any time during the performance period with incipient failure anticipated from the local distress. There are a number of ways that a pavement section can fail.

alligator cracking - Interconnected or interlaced cracks forming a pattern that resembles an alligator's hide. Also, map cracking.

blowup - An upward eruption of a PCC pavement slab near a crack or joint.

crack - A fissure or discontinuity in the pavement surface not necessarily extending through the entire thickness of the pavement.

fatigue cracking - Cracking of the pavement surface as a result of repetitive loading; may be manifested as longitudinal or alligator cracking in the wheel paths for flexible pavement and transverse cracking (and sometimes longitudinal cracking) for jointed concrete pavement.

faulting - Elevation or depression of a PCC slab in relation to an adjoining slab, usually at transverse joints and cracks.

liquefaction - the process of transforming any soil from a solid state to a liquid state, usually as a result of increased pore pressure and reduced shearing resistance (ASTM, 2001). Spontaneous liquefaction may be caused by a collapse of the structure by shock or other type of strain and is associated with a sudden but temporary increase of the prefluid pressure.

longitudinal cracking - Pavement cracking predominantly parallel to the direction of traffic.

pumping - The ejection of foundation material, either wet or dry, through joints or cracks, or along edges of rigid slabs resulting from vertical movements of the slab under traffic, or from cracks in semi-rigid pavements.

punchouts - A broken area of a CRCP bounded by closely spaced cracks usually spaced less than 1 m (3 ft).

random cracking - Uncontrolled and irregular fracturing of a pavement layer.

raveling - A pavement distress characterized by the loss of surface material involving the dislodgment of aggregate particles and degradation of the binder material.

reflective cracking - Cracks in asphalt or concrete surfaces of pavements, occurring over joints or cracks in the underlying layers.

rutting - Longitudinal depression or wearing away of the pavement in wheel paths under load.

spalling - The cracking, breaking, or chipping of pavement edges in the vicinity of a joint or crack.

thermal cracking - Cracks in an asphalt pavement surface, usually full width transverse, as a result of seasonal or diurnal volume changes of the pavement restrained by friction with an underlying layer.

transverse cracking - Pavement cracking predominantly perpendicular to the direction of traffic.

warping - Deformation of a PCC slab caused by a moisture or temperature differential between the upper and lower surfaces.

APPENDIX B: MAIN HIGHWAY PROJECT

B.1 INTRODUCTION

The Main Highway, the example project for the design exercises in this manual, involves the reconstruction and upgrading of an existing county road in the upper northeastern United States. This appendix summarizes the geotechnical and other data available for the design of this project.

A topographic map showing the project horizontal alignment is shown in Figure B-1. The length of the entire reconstruction project is 1.9 miles. The project subsurface investigation consisted of 10 test pits, 10 power auger borings, 5 hand auger borings, and 21 hand rod soundings. Seventeen soil samples were collected and tested in the laboratory.

For the purposes of the design examples in this manual, only a 1500-foot-long subsection of the entire project will be considered. This subsection between Sta. 255+00 and 270+00 is indicated in Figure B-2. Subsequent sections of this appendix summarize the relevant design data for this subsection.

B.2 SUBSURFACE CONDITIONS

A soils map for the site is shown in Figure B-2. Detailed plan and profile views of the subsection of interest are shown in Figure B-3 through Figure B-6. The locations of the various borings, test pits, and soundings are also indicated on these figures. Logs from borings within the subsection of interest are summarized in Figure B-7 through Figure B-14. Similar observations from the test pits are provided in Figure B-15 through Figure B-17.

B.2.1 General Conditions

The dominant soil type along this project is anticipated to consist of clay silt, with some remnants of the existing base material from the existing county road. The natural subgrade soils are all plastic and have been classified as an AASHTO A-4 or A-6 soil, with a frost rating of IV or III. Several undesirable soil conditions may be encountered along this project. These include soil support, drainage, and slumping.

Soil Support. The group index value is used as a general guide to the load bearing characteristics of a soil. It is a function of the liquid limit, plasticity index, and the amount of material passing the No. 200 sieve. Zero indicates good subgrade material, whereas a group index value of 20 or more indicates a poor subgrade material. Group index values obtained on the moist clay silts along this project ranged between 16.3 and 27.3, with an average of 21.6. CBR tests run on three samples (3779[1], 90021, 90022[1]) produced soil support values of 2.6, 2.8, and 2.2, respectively. Based on this, it is anticipated that these soils will have a low bearing capacity resulting in poor soil support. It is anticipated that a standard 30 inch section will not provide adequate structural support. Due to the poor bearing capacity of the clay soils, it has been determined that an 18 inch lift of granular material will be required in addition to the structural aggregate base member. This 18 inch granular lift will provide a working platform for construction equipment to operate upon.

Drainage. Drainage of base and subgrade soils is extremely important along this project due to the presence of clay silt soils along the subgrade. The strength of the clay soils along this project will be dramatically affected by the presence of water. These soils have a high volume change between wet and dry states and will shrink and swell with changes in moisture content. Clay silt soils have a high dry strength but lose much of this strength upon absorbing water. Unfortunately, these soils are poorly drained and may absorb water by capillary action, resulting in low bearing capacity.

Slumping. The clay silt soils along this project have a tendency to slump with slopes greater than 2:1. At times, depending on seasonal conditions, these soils may even slump at slopes shallower than 2:1. It is anticipated that stone ditch protection and stone protection of some backslopes will be required. Vegetation of all exposed soil areas will be very important.

Additional relevant comments and observations from the geotechnical investigation report for this project are as follows:

- It is recommended that construction activities requiring heavy equipment operation on the native subgrade soils not be attempted during early to mid spring due to anticipated moist, soft soils.
- No substantial embankments or cuts are proposed along the project. However, some small embankments or cuts (less than about 5 ft) are proposed over/through the clay and silty clay native soils.

[1] From locations outside of study section.

- Bedrock was not encountered along the project and thus no rock excavation or shallow rock conditions are anticipated.

B.2.2 Detailed Conditions along Study Section (Sta. 255+00 to 270+00)

This section consists of several cut and fill areas with a maximum cut of 4.0 feet and a maximum fill of 3.4 feet along the proposed centerline. Field explorations within this section consist of 2 power auger borings, 3 test pits, and 3 hand auger borings.

Earth excavation is anticipated to consist of existing base material (403) and clay silt (90021, 3777). The base material is classified as an AASHTO A-1-b soil, with a frost class of I. The clay silts are plastic and have been classified as A-6 soils, with a frost class of III. Depending on field conditions with respect to moisture, the clay silts along the surface may be moist to wet and softer than the underlying clay silts.

Subgrade soils are anticipated to consist of existing base material (403) and clay silt (90021, 3777). It is anticipated that most of the clay silts encountered at subgrade within the proposed cut sections may be a little firmer than the overlying clay silts. However, this is based upon the seasonal conditions at the time of the field explorations, and these conditions could change dramatically by the time of construction.

As previously discussed in other sections, drainage is extremely important with respect to the load bearing characteristics of the clay silt subgrade soils. The existing poor pavement conditions in this section can be attributed to the lack of adequate drainage and poor subgrade soils. It has been recommended that all proposed ditching be deepened to 2 feet below proposed subgrade wherever possible.

B.2.3 Laboratory Test Data

The sample log for all test specimens along the study subsection (Sta. 255+00 to 270+00) is given in Table B-1. The following laboratory test information is available:

- Gradation curves for several samples, including some from outside the study section (Figure B-18 and Figure B-19).
- Compaction curves and CBR value for the clayey silt subgrade (Figure B-20 and Figure B-21).
- Compaction curves for the granular working pad (Figure B-22).

- Laboratory resilient modulus values for the clayey silt subgrade (Table B-2) and the granular base material (Table B-3). Table B-4 summarizes the corresponding stress-dependent resilient modulus parameters k_1, k_2, and k_3 for the NCHRP 1-37A level 1 M_R relation:

$$M_R = k_1 p_a \left(\frac{\theta}{p_a}\right)^{k_2} \left(1 + \frac{\tau_{oct}}{p_a}\right)^{k_3}$$

in which

M_R = subgrade resilient modulus (same units as p_a)

θ = bulk stress = $\sigma_1+\sigma_2+\sigma_3$ (same units as p_a)

τ_{oct} = octahedral shear stress = $\frac{1}{3}\sqrt{(\sigma_1-\sigma_2)^2+(\sigma_2-\sigma_3)^2+(\sigma_1-\sigma_3)^2}$ (same units as p_a)

p_a = atmospheric pressure (to make equation dimensionless)

k_1,k_2,k_3 = material properties with constraints $k_1>0$, $k_2 \geq 0$, $k_3 \leq 0$

B.2.4 Field Test Data

Several sets of FWD tests were performed at various times during the years prior to construction. These tests were used to backcalculate the subgrade resilient modulus and pavement effective modulus. These are summarized in Table B-5. The subgrade resilient modulus M_R is calculated from the FWD results using the standard AASHTO equations:

$$M_R = \frac{0.24P}{d_r r}$$

$$r \geq 0.7a_e$$

$$a_e = \sqrt{\left[a^2 + \left(D\sqrt[3]{\frac{E_p}{M_R}}\right)^2\right]}$$

in which

M_R = back-calculated subgrade resilient modulus (psi)

P = applied load (pounds)

d_r = deflection at a distance r from the center of the load (inches)

r = distance from the center of the load (inches)

a_e = radius of the stress bulb at the subgrade-pavement interface (inches)

a = load plate radius (inches)

D = total pavement thickness above the subgrade (inches)

The effective pavement modulus E_p is determined from:

$$d_0 = 1.5pa\left\{\frac{1}{M_R\left[\sqrt{1+\left(\frac{D}{a}\sqrt[3]{\frac{E_p}{M_R}}\right)^2}\right]} + \frac{1-\frac{1}{\sqrt{1+\left(\frac{D}{a}\right)^2}}}{E_p}\right\}$$

in which

d_0 = deflection measured at the center of the load plate, adjusted to a standard temperature of 68°F (inches)

p = load plate pressure (psi)

and the other terms as previously defined. Subgrade resilient modulus for design purposes is usually less than the value back-calculated from FWD data. The AASHTO design guide recommends a design subgrade modulus equal to 33% of the FWD value for flexible pavements and 25% of the FWD value for rigid pavements.

B.3 ENVIRONMENTAL CONDITIONS

The site is located in the northern United States in a cold and wet climate. Freezing Index and frost penetration estimates for the project site are summarized in Table B-6.

B.4 TRAFFIC

Traffic estimates for use in design are summarized in Table B-7. The average traffic level is approximately 750 ESALs per day.

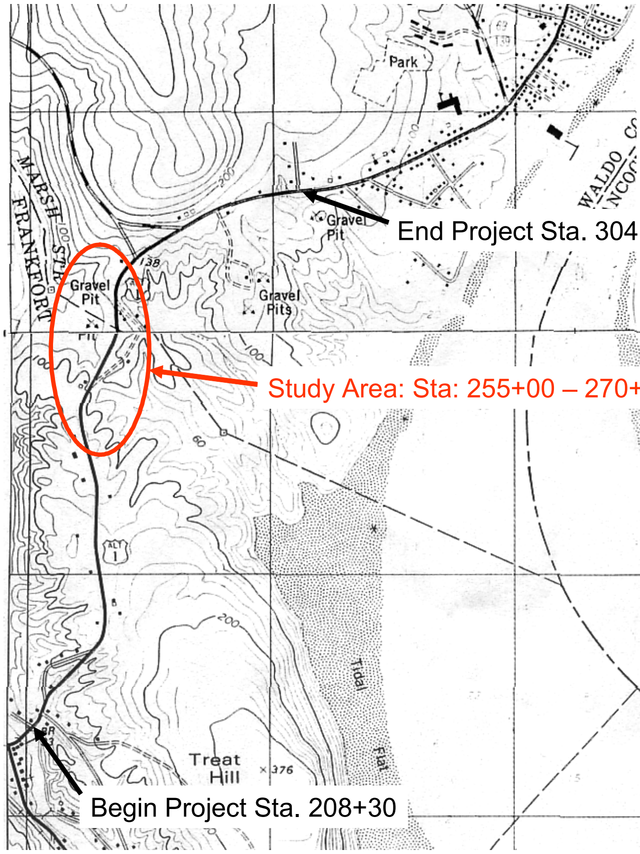


Figure B-1. Project alignment.

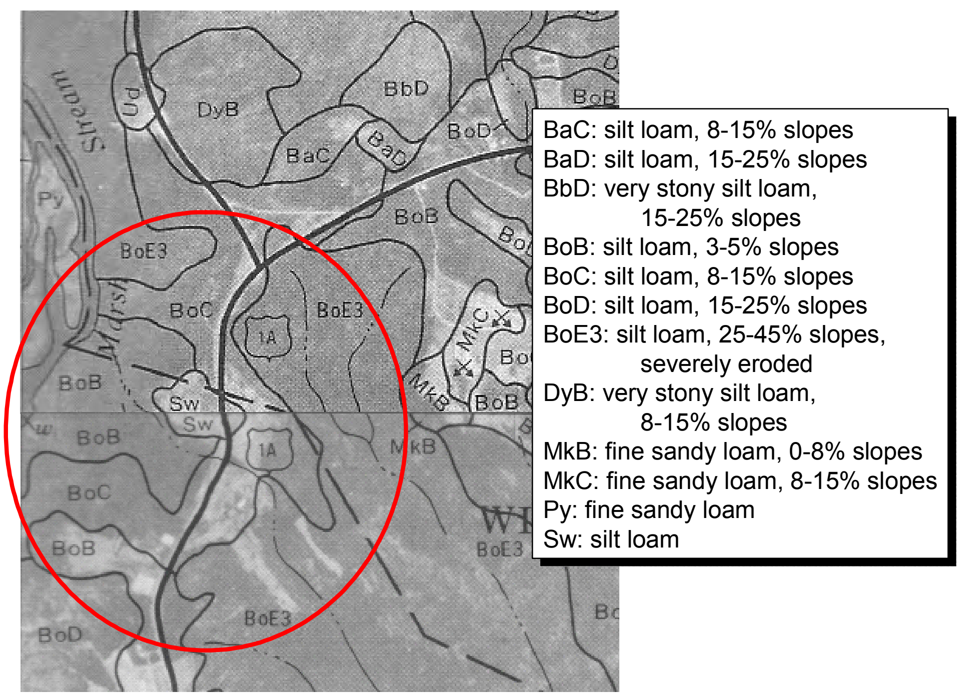


Figure B-2. Soils map for project site.

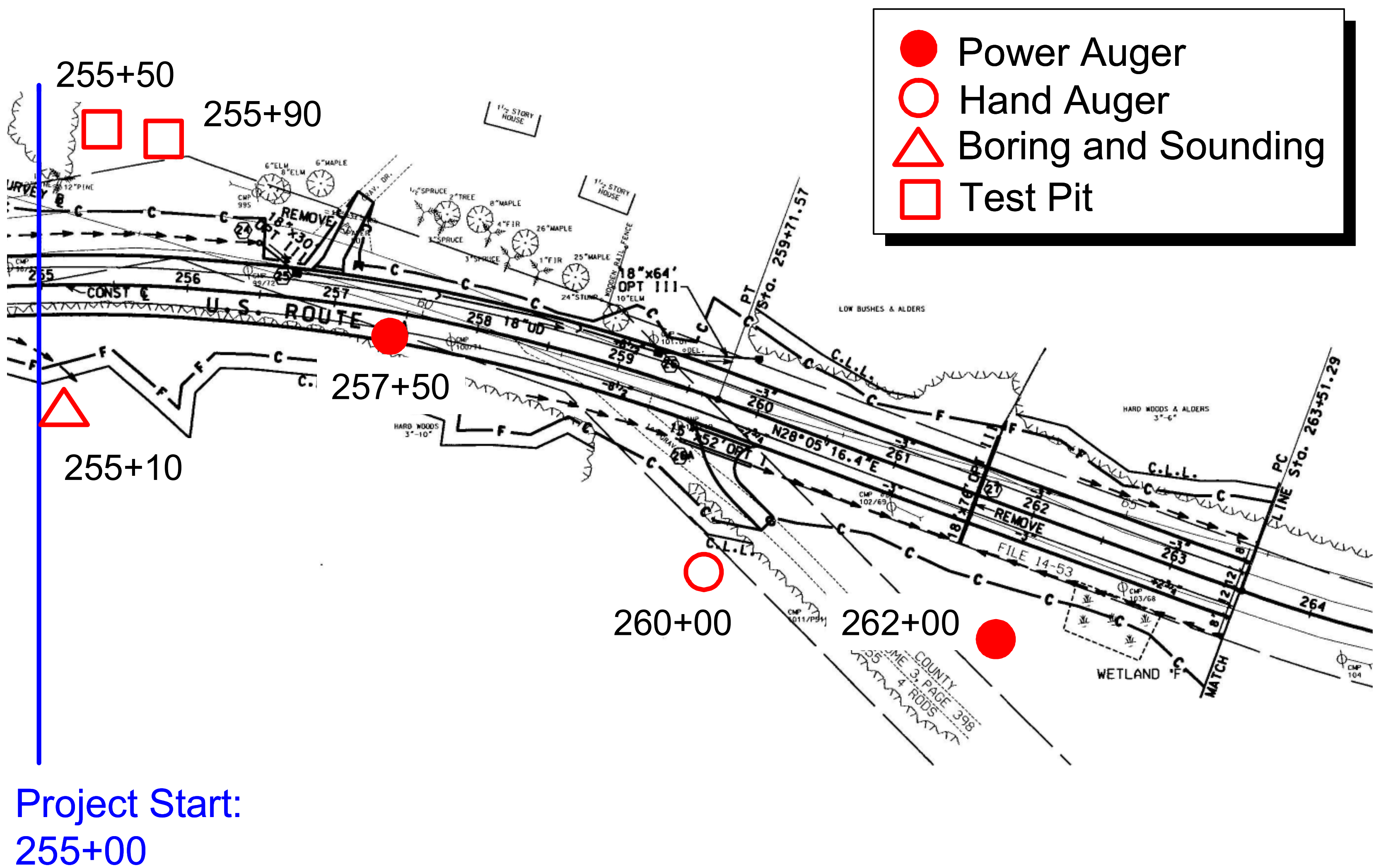


Figure B-3. Horizontal alignment with subsurface exploration locations: Sta. 255+00 to 263+50.

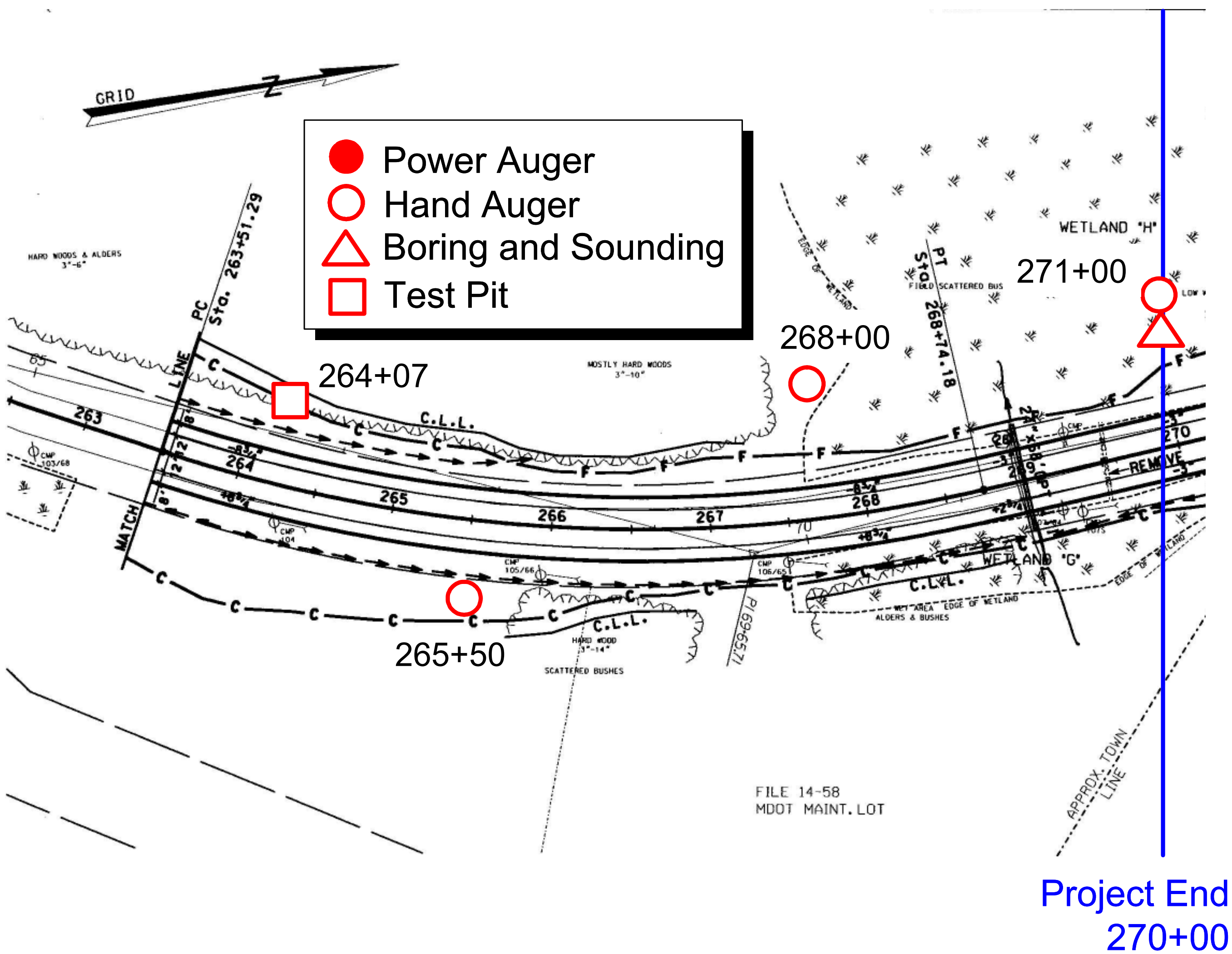


Figure B-4. Horizontal alignment with subsurface exploration locations: Sta. 263+50 to 270+00.

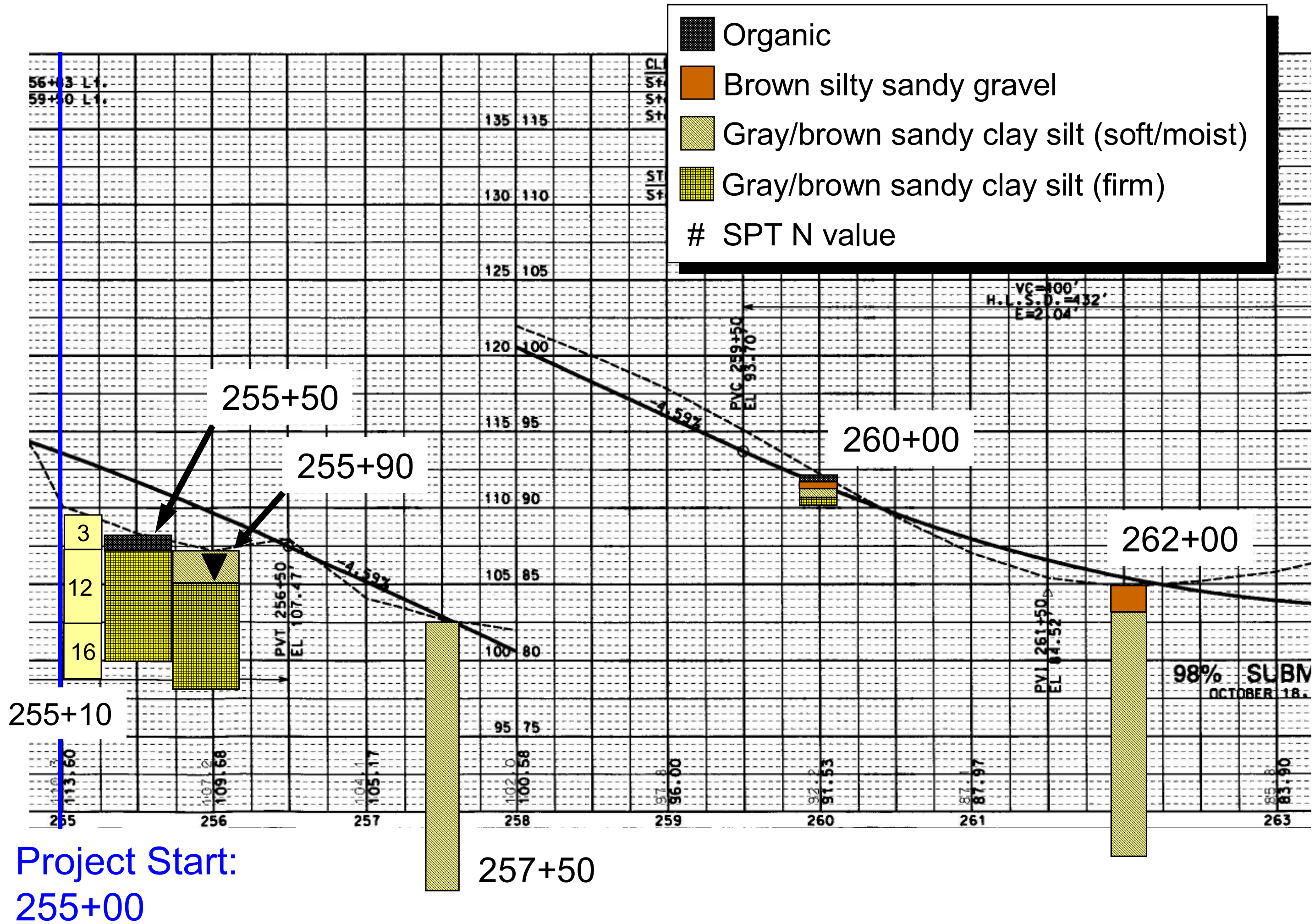


Figure B-5. Vertical alignment with subsurface exploration findings: Sta. 255+00 to 263+50.

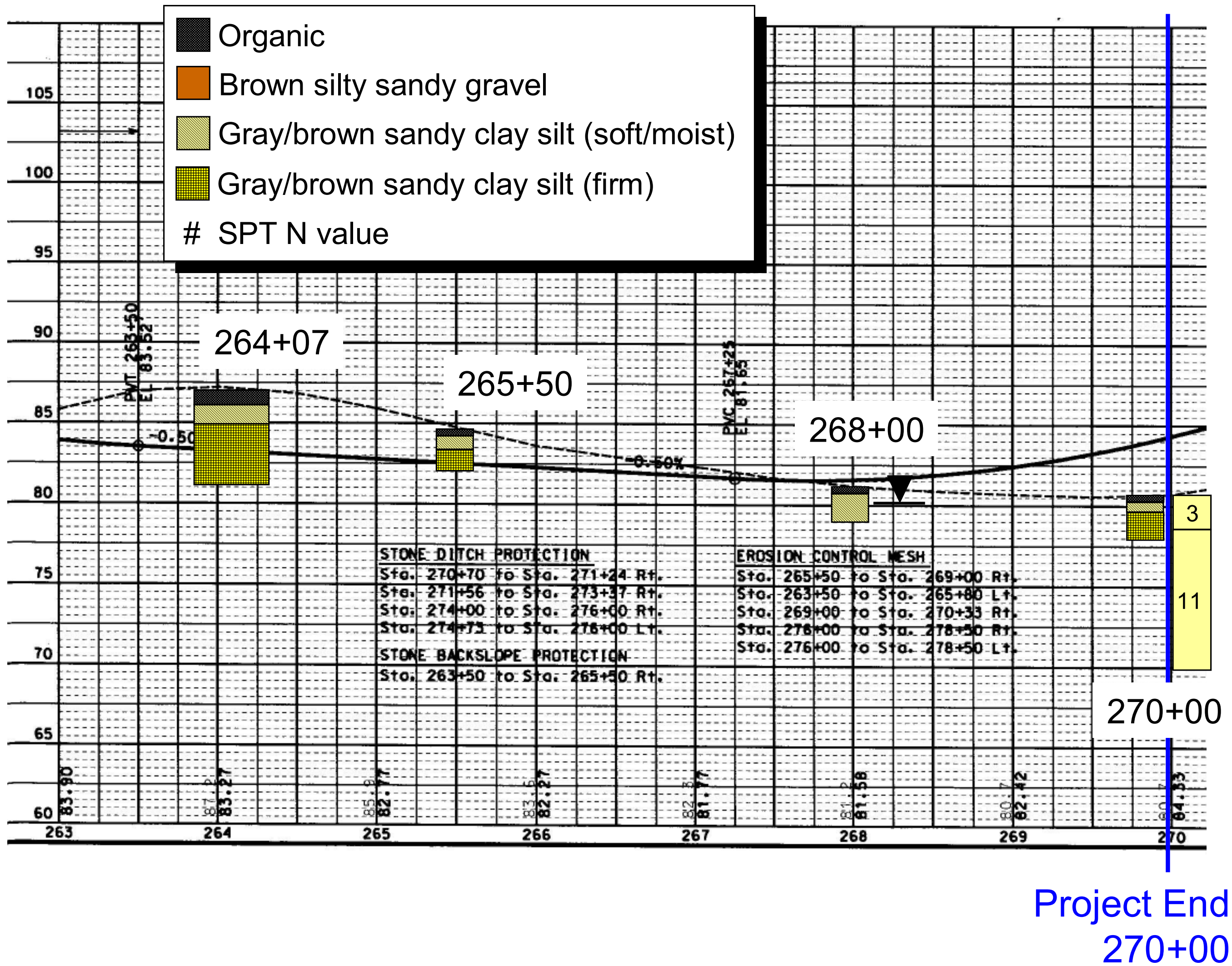


Figure B-6. Vertical alignment with subsurface exploration findings: Sta. 263+50 to 270+00.

Project: Main Highway Project Location: Hometown Project Number: FHWA NHI 132040	**Log of Boring 255+10** Sheet 1 of 1

Date(s) Drilled 5/10/95	Logged By Joe Engineer	Checked By Jane P. Manager
Drilling Method Boring and Sounding	Drill Bit Size/Type	Total Depth of Borehole 12
Drill Rig Type	Drilling Contractor	Approximate Surface Elevation 109.5
Groundwater Level and Date Measured	Sampling Method(s) None	Hammer Data
Borehole Backfill	Location Station 255+10, Offset 30R	

Elevation, feet	Depth, feet	Sample Type	Sample Number	Sampling Resistance, blows/foot	USCS Symbol	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	REMARKS AND OTHER TESTS
110	0			2					Note: Advacent bank washed out below culvert.
				6					
				14					
				12					
				14					
105	5			12					
				12					
				15					
				16					
				16					
100	10			16					
				27					
							Bottom of Boring at 12 feet bgs. No refusal.		
95	15								
90	20								
85	25								
80	30								

Figure B-7. Boring log, station 255+10.

Project: Main Highway Project Location: Hometown Project Number: FHWA NHI 132040	Log of Boring 257+50 Sheet 1 of 1

Date(s) Drilled 5/10/95	Logged By Joe Engineer	Checked By Jane P. Manager
Drilling Method Power Auger	Drill Bit Size/Type	Total Depth of Borehole 19.5 feet bgs
Drill Rig Type	Drilling Contractor	Approximate Surface Elevation 102.5
Groundwater Level and Date Measured	Sampling Method(s) Bulk	Hammer Data
Borehole Backfill	Location Station 257+50, Offset 5R	

Elevation, feet	Depth, feet	Sample Type	Sample Number	Sampling Resistance, blows/foot	USCS Symbol	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	REMARKS AND OTHER TESTS
102	0				A-6		Moist brown sandy clay silt		
97	5								
			90021					24	
92	10								
87	15								
82	20						Bottom of Boring at 19.5 feet bgs. No refusal.		
77	25								
72	30								

Figure B-8. Boring log, station 257+50.

Project: Main Highway Project Location: Hometown Project Number: FHWA NHI 132040	**Log of Boring 260+00** Sheet 1 of 1

Date(s) Drilled 7/11/95	Logged By Joe Engineer	Checked By Jane P. Manager
Drilling Method Hand Auger	Drill Bit Size/Type	Total Depth of Borehole 1.9
Drill Rig Type	Drilling Contractor	Approximate Surface Elevation 92
Groundwater Level and Date Measured	Sampling Method(s) None	Hammer Data
Borehole Backfill	Location Station 260+00, Offset 40R	

Elevation, feet	Depth, feet	Sample Type	Sample Number	Sampling Resistance, blows/foot	USCS Symbol	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	REMARKS AND OTHER TESTS
92	0				A-7-6		Moist rooted organic matter		
					A-4		Moist rooted silty pebbly sand		
					A-4		Rooted moist gray clay silt w/ some organics (similar to sample 232)		
							Moist mottled sandy silt (similar to sample 233)		
							Firm moist mottled clay silt (similar to sample 233)		
							Bottom of Boring at 1.9 feet bgs. No refusal.		
87	5								
82	10								
77	15								
72	20								
67	25								
62	30								

Figure B-9. Boring log, station 260+00.

Project: Main Highway Project Location: Hometown Project Number: FHWA NHI 132040	Log of Boring 262+00 Sheet 1 of 1

Date(s) Drilled 5/10/95	Logged By Joe Engineer	Checked By Jane P. Manager
Drilling Method Power Auger	Drill Bit Size/Type	Total Depth of Borehole 19.5
Drill Rig Type	Drilling Contractor	Approximate Surface Elevation 85
Groundwater Level and Date Measured	Sampling Method(s) None	Hammer Data
Borehole Backfill	Location Station 262+00, Offset 30R	

Elevation, feet	Depth, feet	Sample Type	Sample Number	Sampling Resistance, blows/foot	USCS Symbol	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	REMARKS AND OTHER TESTS
85	0						Brown silty sandy gravel		
					A-4		Moist gray brown sandy clay silt (similar to sample 3777)		
80	5								
75	10								
70	15								
65	20						Bottom of Boring at 19.5 feet bgs. No refusal.		
60	25								
55	30								

Figure B-10. Boring log, station 262+00.

Project: Main Highway Project Location: Hometown Project Number: FHWA NHI 132040	**Log of Boring 265+50** Sheet 1 of 1

Date(s) Drilled 7/11/95	Logged By Joe Engineer	Checked By Jane P. Manager
Drilling Method Hand Auger	Drill Bit Size/Type	Total Depth of Borehole 2.3
Drill Rig Type	Drilling Contractor	Approximate Surface Elevation 84.5
Groundwater Level and Date Measured 1.3 bgs	Sampling Method(s) Bulk	Hammer Data
Borehole Backfill	Location Station 265+50, Offset 30R	

Elevation, feet	Depth, feet	Sample Type	Sample Number	Sampling Resistance, blows/foot	USCS Symbol	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	REMARKS AND OTHER TESTS
84	0		232		A-7-5		Moist rooted organic matter.	92	
			234		A-4		Moist rooted clay silt with some organics. (ATD)		
					A-4		Moist pebbly sandy clay silt.		
							Moist mottled sandy gray silt (similar to sample 233).		
							Bottom of Boring at 2.3 feet bgs. No refusal.		
79	5								
74	10								
69	15								
64	20								
59	25								
54	30								

Figure B-11. Boring log, station 265+50.

Project: Main Highway **Project Location: Hometown** **Project Number: FHWA NHI 132040**	**Log of Boring 268+00** Sheet 1 of 1

Date(s) Drilled **7/11/95**	Logged By **Joe Engineer**	Checked By **Jane P. Manager**
Drilling Method **Hand Auger**	Drill Bit Size/Type	Total Depth of Borehole **2.8**
Drill Rig Type	Drilling Contractor	Approximate Surface Elevation **81**
Groundwater Level and Date Measured	Sampling Method(s) **None**	Hammer Data
Borehole Backfill	Location **Station 268+00, Offset 30L**	

Elevation, feet	Depth, feet	Sample Type	Sample Number	Sampling Resistance, blows/foot	USCS Symbol	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	REMARKS AND OTHER TESTS
81	0				A-7-5		Moist rooted organic matter		
					A-4		Rooted moist gray clay silt (less organic than sample 232)		
							Firm moist mottled sandy clay silt (similar to sample 233)		
							Bottom of Boring at 2.8 feet bgs. No refusal.		
76	5								
71	10								
66	15								
61	20								
56	25								
51	30								

Figure B-12. Boring log, station 268+00.

Project: Main Highway Project Location: Hometown Project Number: FHWA NHI 132040	**Log of Boring 270+00** Sheet 1 of 1

Date(s) Drilled 7/11/95	Logged By Joe Engineer	Checked By Jane P. Manager
Drilling Method Hand Auger	Drill Bit Size/Type	Total Depth of Borehole 2.4
Drill Rig Type	Drilling Contractor	Approximate Surface Elevation 80.5
Groundwater Level and Date Measured	Sampling Method(s) Bulk	Hammer Data
Borehole Backfill	Location Station 270+00, Offset 35L	

Elevation, feet	Depth, feet	Sample Type	Sample Number	Sampling Resistance, blows/foot	USCS Symbol	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	REMARKS AND OTHER TESTS
80	0				A-7-5		Moist rooted organic matter.		
					A-4		Rooted moist gray clay silt w/ some organics (similar to sample 232).		
			233				Mottled moist clay silt (firm)	23	
							Bottom of Boring at 2.4 feet bgs. No refusal.		
75	5								
70	10								
65	15								
60	20								
55	25								
50	30								

Figure B-13. Boring log, station 270+00.

Project: Main Highway Project Location: Hometown Project Number: FHWA NHI 132040	Log of Boring 270+10 Sheet 1 of 1

Date(s) Drilled 7/11/95	Logged By Joe Engineer	Checked By Jane P. Manager
Drilling Method Boring and Sounding	Drill Bit Size/Type	Total Depth of Borehole 12
Drill Rig Type	Drilling Contractor	Approximate Surface Elevation 81
Groundwater Level and Date Measured	Sampling Method(s) None	Hammer Data
Borehole Backfill	Location Station 270+10, Offset 30L	

Elevation, feet	Depth, feet	Sample Type	Sample Number	Sampling Resistance, blows/foot	USCS Symbol	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	REMARKS AND OTHER TESTS
81	0			2					
				4					
				12					
				9					
				10					
76	5			9					
				11					
				9					
				14					
				12					
71	10			10					
				9					
							Bottom of Boring at 12 feet bgs. No refusal.		
66	15								
61	20								
56	25								
51	30								

Figure B-14. Boring log, station 270+10.

Project: Main Highway Project Location: Hometown Project Number: FHWA NHI 132040	**Log of Boring 255+50 (Pit)** Sheet 1 of 1

Date(s) Drilled 11/5/92	Logged By Joe Engineer	Checked By Jane P. Manager
Drilling Method Test Pit	Drill Bit Size/Type	Total Depth of Borehole 10 feet bgs
Drill Rig Type	Drilling Contractor	Approximate Surface Elevation 108
Groundwater Level and Date Measured	Sampling Method(s) None	Hammer Data
Borehole Backfill	Location Station 255+50, Offset 30L	

Elevation, feet	Depth, feet	Sample Type	Sample Number	Sampling Resistance, blows/foot	USCS Symbol	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	REMARKS AND OTHER TESTS
108	0						Loose brown sandy clay silt.		
							Firm gray brown clay silt. Note: Soil becomes slightly moist with depth, but no seepage visible in test pit.		
103	5								
98	10						Bottom of test pit at 10 feet bgs.		
93	15								
88	20								
83	25								
78	30								

Figure B-15. Test pit log, station 255+50.

Project: Main Highway Project Location: Hometown Project Number: FHWA NHI 132040	Log of Boring 255+90 (Pit) Sheet 1 of 1

Date(s) Drilled 6/14/90	Logged By Joe Engineer	Checked By Jane P. Manager
Drilling Method Test Pit	Drill Bit Size/Type	Total Depth of Borehole 10 feet bgs
Drill Rig Type	Drilling Contractor	Approximate Surface Elevation 103
Groundwater Level and Date Measured 2.5 bgs	Sampling Method(s) None	Hammer Data
Borehole Backfill	Location Station 255+90, Offset 35L	

Elevation, feet	Depth, feet	Sample Type	Sample Number	Sampling Resistance, blows/foot	USCS Symbol	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	REMARKS AND OTHER TESTS
103	0						Moist soft gray brown clay silt w/some sand, pebbles at shallow depth.		
							(ATD) ∇ Firm gray brown clay silt.		
98	5								
93	10						Bottom of test pit at 10 feet bgs.		
88	15								
83	20								
78	25								
73	30								

Figure B-16. Test pit log, station 255+90.

Project: Main Highway Project Location: Hometown Project Number: FHWA NHI 132040	Log of Boring 264+07 (Pit) Sheet 1 of 1

Date(s) Drilled 6/14/90	Logged By Joe Engineer	Checked By Jane P. Manager
Drilling Method Test Pit	Drill Bit Size/Type	Total Depth of Borehole 7
Drill Rig Type	Drilling Contractor	Approximate Surface Elevation 87
Groundwater Level and Date Measured	Sampling Method(s) Bulk	Hammer Data
Borehole Backfill	Location Station 264+07, Offset 20L	

Elevation, feet	Depth, feet	Sample Type	Sample Number	Sampling Resistance, blows/foot	USCS Symbol	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	REMARKS AND OTHER TESTS
87	0						Rooted clay silt topsoil.		
							Soft moist gray brown clay silt.		
					A-4		Firm gray brown clay silt.		
82	5		3777					24	
							Bottom of test pit at 7 feet bgs.		
77	10								
72	15								
67	20								
62	25								
57	30								

Figure B-17. Test pit log, station 264+07.

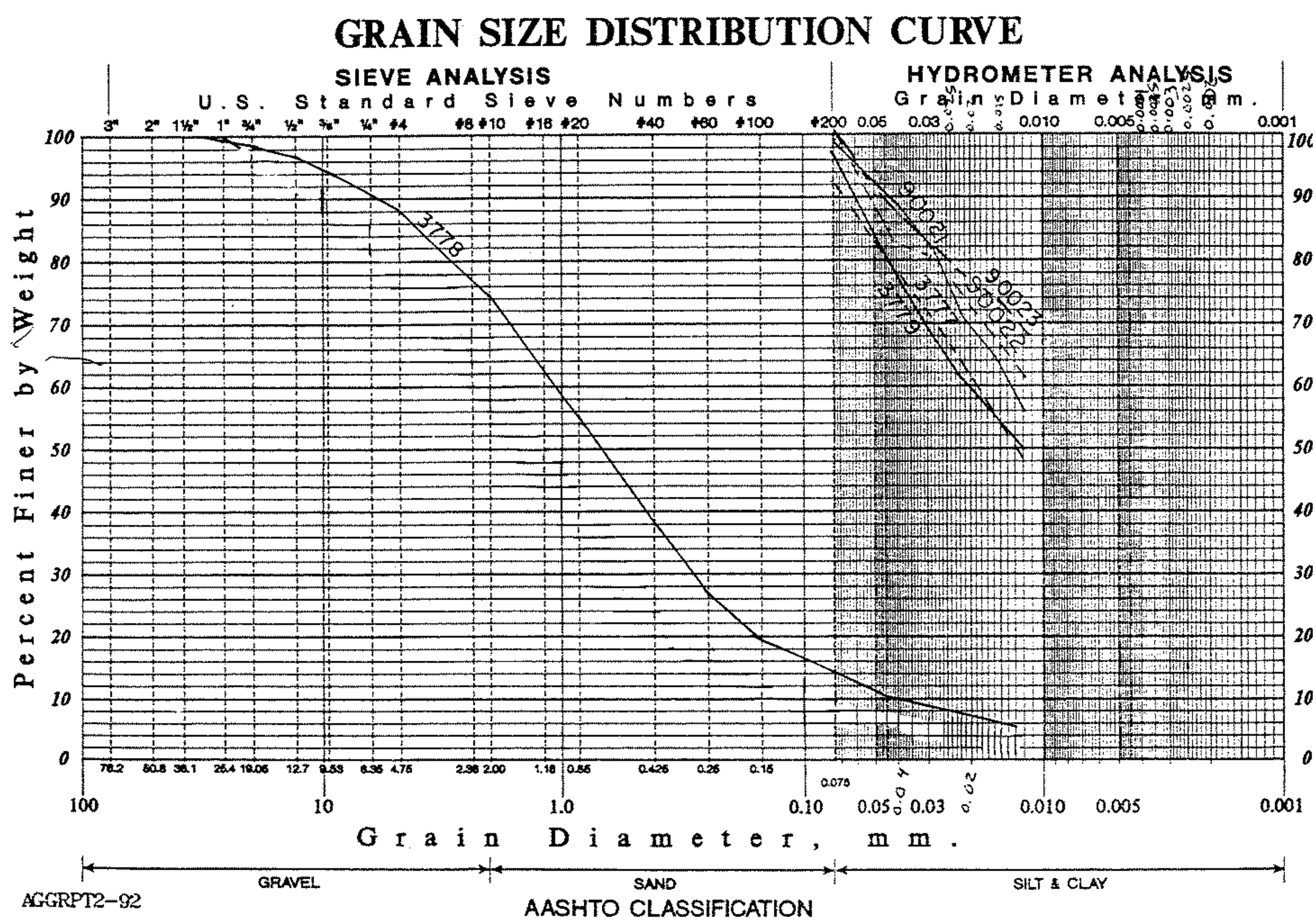


Figure B-18. Grain size distribution curves.

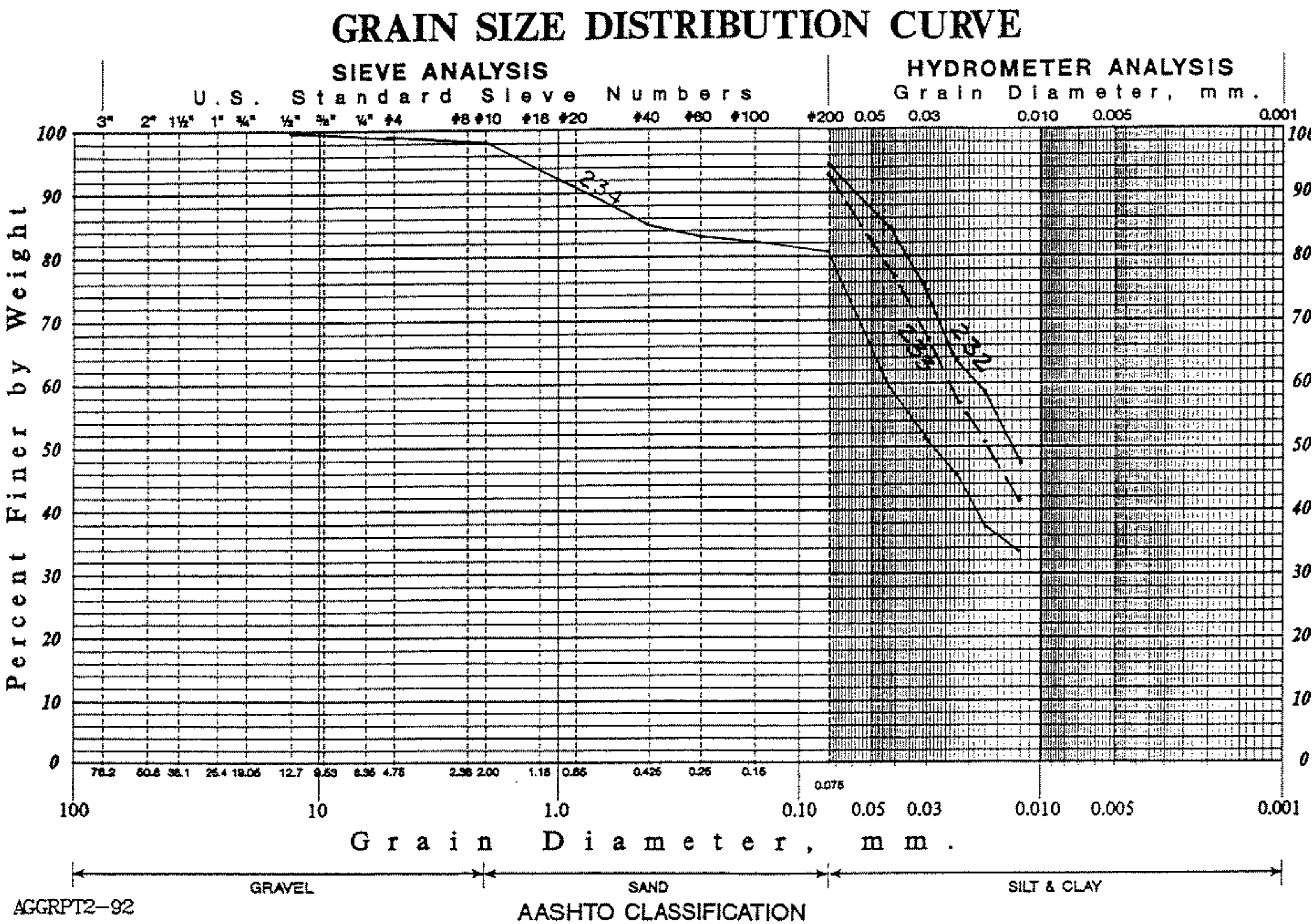


Figure B-19. Grain size distribution curves.

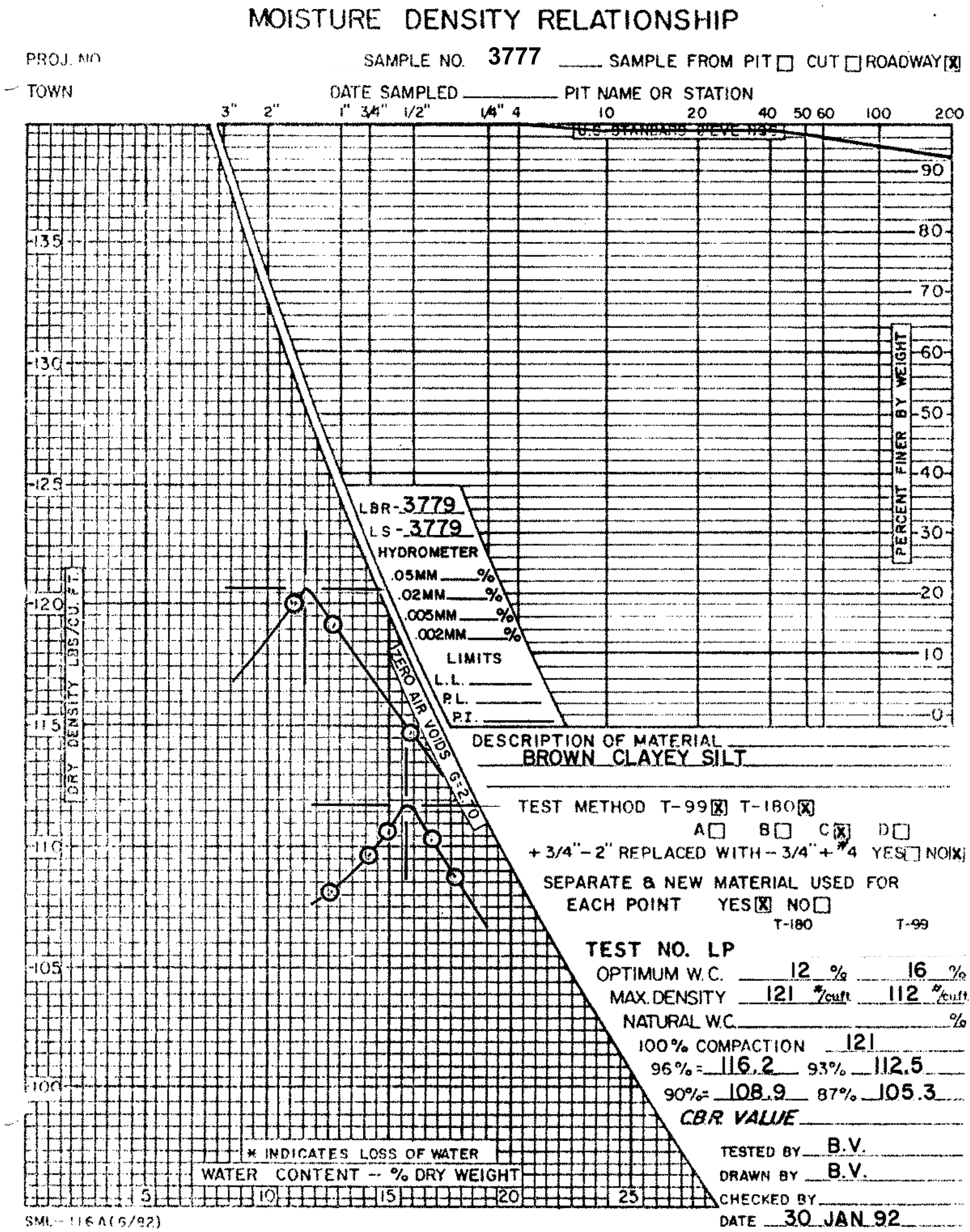


Figure B-20. Compaction curves for clayey silt subgrade soil (sample 90021).

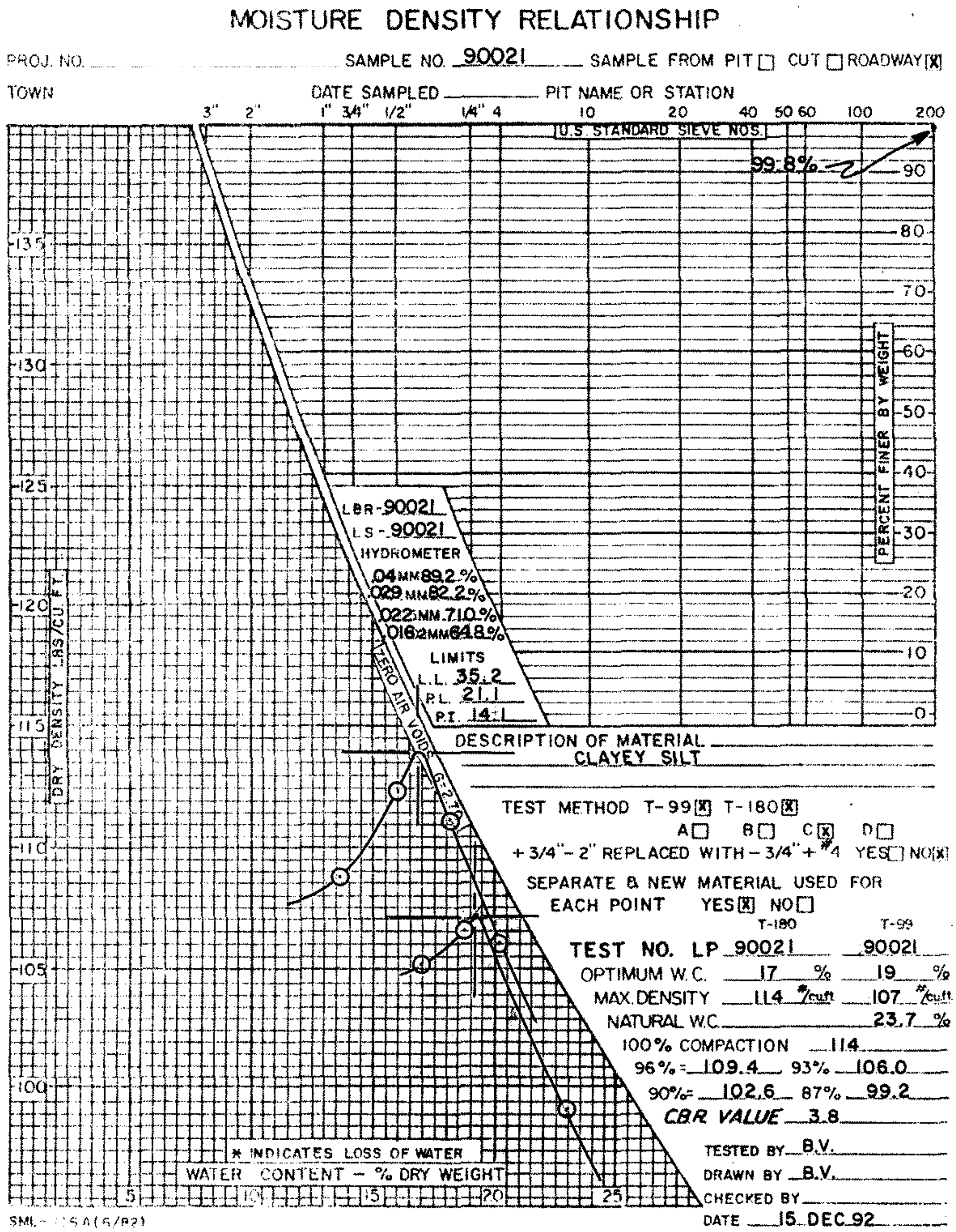


Figure B-21. Compaction curves for clayey silt subgrade soil (sample 90021).

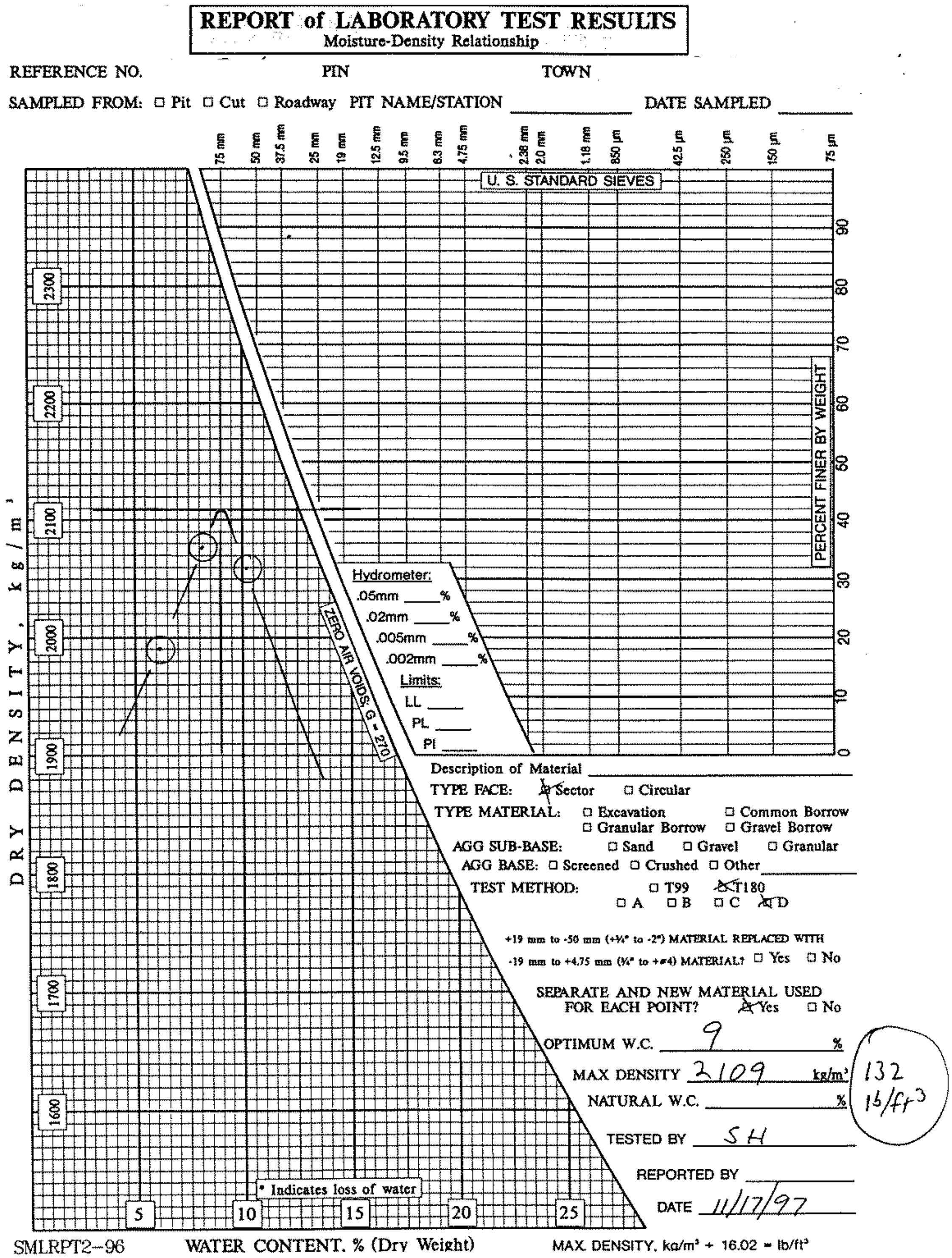


Figure B-22. Compaction curves for granular working platform.

Table B-1. Sample log.

STATION	OFFSET	DEPTH	SAMPLE No.	W.C	L.L.	P.I.	IGN.	pH.	CLASSIFICATION	
									AASHTO	FROST
257+50	5R	8.0 - 9.2	90021	24	35.2	14.1		6.5	A-6	III
264+07	20L	4.0 - 5.0	3777	24	27.5	9.1		6.5	A-4	IV
265+50	30R	0.3 - 0.8	234	24	26.1	5.6			A-4	IV
265+50	30R	0.8 - 1.8	232	92	65.1	29.4	16.6		A-7-5	III
270+00	35L	1.2 - 2.2	233	23	22	6.4			A-4	IV
Classification of these soil samples is in accordance with AASHTO Classification System M-145-40. This Classification is followed by the "Frost Susceptibility Rating"from zero (non-frost susceptible) to Class IV (highly frost susceptible). The 'Frost Susceptible Rating"is based upon the Corps of Engineers Classification Systems.										

Table B-2. Laboratory resilient modulus data for clay silt subgrade (sample 3777).

Replicate	Contact Stress (psi)	Confining Pressure (psi)	Cyclic Stress (psi)	θ (psi)	τ_{oct} (psi)	M_R (psi)
1	0.620	8.032	0.946	25.660	0.738	1450
	0.743	6.099	0.839	19.880	0.746	1417
	0.307	4.107	0.806	13.433	0.525	1320
	0.151	2.043	0.757	7.037	0.428	1152
	0.636	8.032	1.431	26.164	0.974	1094
	0.455	5.969	1.390	19.752	0.870	1046
	0.307	3.972	1.316	13.537	0.765	971
	0.184	2.019	1.209	7.450	0.656	888
2	0.603	8.045	0.880	25.618	0.699	1703
	0.447	6.186	0.880	19.886	0.626	1675
	0.299	4.139	0.831	13.547	0.532	1519
	0.167	2.207	0.773	7.560	0.443	1337
	0.620	8.001	1.464	26.087	0.982	1289
	0.447	6.082	1.431	20.124	0.885	1240
	0.307	4.059	1.324	13.808	0.769	1125
	0.167	2.010	1.225	7.423	0.656	1013
3	0.603	8.088	0.806	25.674	0.664	1351
	0.463	6.030	0.789	19.342	0.591	1341
	0.307	4.036	0.724	13.140	0.486	1177
	0.151	2.028	0.674	6.909	0.389	1046
	0.603	8.025	1.357	26.036	0.924	1025
	0.463	6.083	1.316	20.027	0.839	970
	0.332	4.097	1.250	13.873	0.746	898
	0.200	2.012	1.143	7.381	0.633	808

Table B-3. Laboratory resilient modulus data for granular base material.

Replicate	Contact Stress (psi)	Confining Pressure (psi)	Cyclic Stress (psi)	θ (psi)	τ_{oct} (psi)	M_R (psi)
1	0.60	3.07	0.72	10.55	0.62	17623
	1.17	5.96	2.51	21.55	1.74	19768
	2.04	9.98	4.89	36.87	3.27	27007
	3.07	15.08	8.07	56.36	5.25	34378
	4.02	20.08	11.35	75.60	7.25	43204
	0.60	3.06	2.17	11.96	1.31	10960
	1.16	6.00	5.54	24.69	3.16	16819
	2.04	9.97	10.54	42.48	5.93	23760
	3.00	14.91	17.48	65.21	9.66	33003
	4.01	20.02	23.99	88.07	13.20	42197
	0.61	3.03	4.84	14.55	2.57	10496
	1.20	6.07	12.30	31.71	6.36	17847
	2.04	10.03	23.44	55.58	12.01	27008
	3.03	14.98	35.69	83.66	18.25	37326
	4.05	20.10	46.70	111.03	23.92	44990
	0.63	2.97	7.56	17.09	3.86	10734
	1.19	6.06	20.15	39.52	10.06	20036
	2.06	10.00	35.35	67.41	17.64	28956
	3.02	15.04	51.98	100.11	25.93	37193
	4.01	19.99	67.65	131.64	33.78	44559
	0.61	2.99	14.79	24.37	7.26	12506
	1.22	5.95	35.32	54.39	17.23	21959
	2.08	10.03	58.14	90.30	28.38	29538
	3.00	15.04	84.55	132.69	41.27	38762
	4.05	20.03	108.67	172.83	53.14	49332
2	0.61	3.02	1.29	10.97	0.90	7386
	1.23	6.03	2.63	21.96	1.82	11375
	2.03	10.04	4.91	37.06	3.27	20089
	3.00	15.02	7.74	55.82	5.07	28547
	4.03	20.06	10.84	75.04	7.01	38724
	0.50	3.06	2.59	12.26	1.46	6802
	1.21	6.06	5.89	25.28	3.35	12349
	2.08	10.01	10.13	42.23	5.75	20448
	3.06	15.04	16.02	64.20	9.00	30563
	4.08	19.93	21.30	85.17	11.96	39487
	0.59	3.03	4.90	14.57	2.59	7355
	1.21	5.85	11.73	30.48	6.10	13987
	2.00	10.00	21.36	53.37	11.01	23784

Replicate	Contact Stress (psi)	Confining Pressure (psi)	Cyclic Stress (psi)	θ (psi)	τ_{oct} (psi)	M_R (psi)
	3.04	15.05	32.28	80.49	16.65	34193
	4.02	20.06	42.26	106.47	21.82	41623
	0.60	3.02	7.81	17.46	3.97	8253
	1.20	6.06	18.79	38.16	9.42	16909
	2.01	10.02	32.66	64.72	16.34	26331
	3.03	14.93	47.85	95.66	23.98	35554
	4.06	20.01	62.46	126.55	31.36	43168
	0.61	2.97	14.35	23.88	7.05	11096
	1.21	6.05	32.38	51.73	15.84	20945
	2.02	9.93	53.30	85.12	26.08	30242
	3.02	15.02	76.57	124.65	37.52	39709
	4.09	19.97	101.06	165.06	49.57	51042

Table B-4. Stress-dependent resilient modulus parameters.

Soil	k_1	k_2	k_3
Clay silt subgrade	170	0.450	-16.388
Granular base	662	1.010	-0.585

Table B-5. Field backcalculated values for subgrade modulus and effective pavement modulus.

Section	M_R (psi)	E_p (psi)
255+00 - 261+50	3000	25714
261+50 - 268+00	3857	27142
268+00 - 269+00	2571	27142

Table B-6. Freezing index and frost penetration estimates (assuming 32 inches of pavement and base).

	Total Frost Penetration		Frost Penetration into Subgrade	
Freezing Index	Nongranular Subgrade	Granular Subgrade	Nongranular Subgrade	Granular Subgrade
Mean 1200	46 in.	65 in.	14 in.	33 in.
Design 1700	53 in.	85 in.	21 in.	15 in.

Table B-7. Design traffic.

Vehicle Class	Est. AADT (2 way)	Est. AADT Percentage	ESAL Factor	Design ESALs
4	90	1.51%	0.4700	21.15
5	297	4.99%	0.3000	44.55
6	361	6.04%	1.3300	240.7
7	111	1.86%	3.3600	186.48
8	118	1.96%	0.8600	50.74
9	127	2.13%	1.0900	69.22
10	79	1.32%	2.8800	113.76
11	0	0.00%	1.0000	0.00
12	0	0.00%	1.0000	0.00
13	0	0.00%	3.7500	0.00
Light Vehicles	4727	80.19%	0.0008	1.89
All Vehicles	5910	100.00%		727.85

APPENDIX C: 1993 AASHTO DESIGN METHOD

C.1 INTRODUCTION

The *AASHTO Guide for Design of Pavement Structures* (AASHTO, 1993) is the primary document used to design new and rehabilitated highway pavements. Approximately 80% of all states use the AASHTO pavement design procedures, with the majority using the 1993 version. All versions of the AASHTO Design Guide are empirical design methods based on field performance data measured at the AASHO Road Test in 1958-60.

Chapter 3 of this manual describes the evolution of the various versions of the AASHTO Design Guide. Geotechnical inputs to the 1993 AASHTO design procedure are detailed in Chapter 5. Chapter 6 provides some design examples using the 1993 AASHTO procedures.

The overall approach of the 1993 AASHTO procedure for both flexible and rigid pavements is to design for a specified serviceability loss at the end of the design life of the pavement. Serviceability is defined in terms of the Present Serviceability Index, *PSI*, which varies between the limits of 5 (best) and 0 (worst). Serviceability loss at end of design life, ΔPSI, is partitioned between traffic and environmental effects, as follows (see also Figure 3.8):

$$\Delta PSI = \Delta PSI_{TR} + \Delta PSI_{SW} + \Delta PSI_{FH} \tag{C.1}$$

in which ΔPSI_{TR}, ΔPSI_{SW} and ΔPSI_{FH} are the components of serviceability loss attributable to traffic, swelling, and frost heave, respectively. The structural design procedures for swelling and frost heave are the same for both flexible and rigid pavements; these are detailed in Appendix G of the 1993 AASHTO Guide. The structural design procedures for traffic are different for flexible and rigid pavement types. These procedures are summarized below in Sections C.2 and C.3, respectively. For simplicity, only the design procedures for new construction are summarized here. The design procedures for reconstruction are similar, except that characterization of recycled materials may be required. See the 1993 AASHTO Guide for details of additional procedures (*e.g.,* determination of remaining structural life for overlay design) relevant to rehabilitation design.

C.2 FLEXIBLE PAVEMENT STRUCTURAL DESIGN

Design Equation

The empirical expression relating traffic, pavement structure, and pavement performance for flexible pavements is:

$$\log_{10}\left(W_{18}\right) = Z_R S_0 + 9.36\log_{10}\left(SN+1\right) - 0.20 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.2-1.5}\right]}{0.40+\frac{1094}{\left(SN+1\right)^{5.19}}} + 2.32\log_{10}\left(M_R\right) - 8.07 \qquad \text{(C.2)}$$

in which:

W_{18}	=	number of 18 kip equivalent single axle loads (ESALs)
Z_R	=	standard normal deviate (function of the design reliability level)
S_0	=	overall standard deviation (function of overall design uncertainty)
ΔPSI	=	allowable serviceability loss at end of design life
M_R	=	subgrade resilient modulus
SN	=	structural number (a measure of required structural capacity)

The first five parameters typically are the inputs to the design equation, and SN is the output. Equation (C.2) must be solved implicitly for the structural number SN as a function of the input parameters. The structural number SN is defined as:

$$SN = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3 \qquad \text{(C.3)}$$

in which D_1, D_2, and D_3 are the thicknesses (inches) of the surface, base, and subbase layers, respectively, a_1, a_2, and a_3 are corresponding structural layer coefficients, and m_2 and m_3 are drainage coefficients for the base and subbase layers, respectively. Equation (C.3) can be generalized for additional bound and/or unbound layers. Note that there may be many combinations of layer thicknesses that can provide satisfactory SN values; cost and other issues must be considered to determine the optimal final design.

Analysis Period

Performance period refers to the time that a pavement design is intended to last before it needs rehabilitation. It is equivalent to the time elapsed as a new, reconstructed, or rehabilitated pavement structure deteriorates from its initial serviceability to its terminal serviceability. The term "analysis period" refers to the overall duration that the design strategy must cover. It may be identical to the performance period. However, realistic performance limitations may require planned rehabilitation within the desired analysis period, in which case, the analysis period may encompass multiple performance periods. Analysis period in this context is synonymous with design life in the 1993 AASHTO Guide. AASHTO recommendations for analysis periods for different types of roads are summarized in Table C-1.

Table C-1. Guidelines for length of analysis period (AASHTO, 1993).

Highway conditions	Analysis period (years)
High-volume urban	30 – 50
High-volume rural	20 – 50
Low-volume paved	15 – 25
Low-volume aggregate surface	10 – 20

Traffic

Traffic is one of the most important factors in pavement design, and every effort should be made to collect accurate data specific to each project. Traffic analysis requires the evaluation of initial traffic volume, traffic growth, directional distribution, and traffic type.

The AASHTO Design Guide is based on cumulative 18 kip (80 KN) equivalent single-axle loads (ESALs). Detailed traffic analysis is beyond the scope of this reference manual. However, ESALs may be estimated using the following equation:

$$ESAL = (ADT_0)(T)(T_f)(G)(D)(L)(365)(Y) \qquad \text{(C.4)}$$

in which:

ADT_0 = average daily traffic at the start of the design period

T = percentage of trucks in the ADT

T_f = truck factor, or the number of 18 kip ESALs per truck

G	=	traffic growth factor
D	=	directional distribution factor
L	=	lane distribution factor
Y	=	design period in years

AASHTO (1993) and standard pavement engineering textbooks (e.g, Huang, 2004) provide details on the determination of all of these parameters and estimation of design ESALs.

Reliability

Design reliability is defined as the probability that a pavement section will perform satisfactorily over the design period. It must account for uncertainties in traffic loading, environmental conditions, and construction materials. The AASHTO design method accounts for these uncertainties by incorporating a reliability level R to provide a factor of safety into the pavement design and thereby increase the probability that the pavement will perform as intended over its design life. The levels of reliability recommended by AASHTO for various classes of roads are summarized in Table C-2.

The reliability level is not included directly in the AASHTO design equations. Rather, it is used to determine the standard normal deviate Z_R. Values of Z_R corresponding to selected levels of reliability are summarized in Table C-3.

The AASHTO design equations also require specification of the overall standard deviation S_0. For flexible pavements, values for S_0 typically range between 0.35 and 0.50, with a value of 0.45 commonly used for design.

Table C-2. Suggested levels of reliability for various functional classifications (AASHTO, 1993).

Functional classification	Recommended level of reliability	
	Urban	Rural
Interstate and other freeways	85 – 99.9	80 – 99.9
Principal arterials	80 – 99	75 – 95
Collectors	80 – 95	75 – 95
Local	50 – 80	50 – 80

Note: Results base on a survey of AASHTO Pavement Design Task Force.

Table C-3. Standard normal deviates for various levels of reliability.

Reliability (%)	Standard normal deviate (Z_R)	Reliability (%)	Standard normal deviate (Z_R)
50	0.000	93	-1.476
60	-0.253	94	-1.555
70	-0.524	95	-1.645
75	-0.674	96	-1.751
80	-0.841	97	-1.881
85	-1.037	98	-2.054
90	-1.282	99	-2.327
91	-1.340	99.9	-3.090
92	-1.405	99.99	-3.750

Serviceability

Serviceability is quantified by the Present Serviceability Index, PSI. Although PSI theoretically ranges between 5 and 0, the actual range for real pavements is between about 4.5 to 1.5.

The initial serviceability index p_o corresponds to road conditions immediately after construction. A typical value of p_o for flexible pavements is 4.2. The terminal serviceability index p_t is defined as the lowest serviceability that will be tolerated before rehabilitation or reconstruction becomes necessary. A terminal serviceability index of 2.5 or higher is recommended for design of major highways. Thus, a typical allowable serviceability loss due to traffic for flexible pavements can be expressed as:

$$\Delta PSI = p_t - p_o = 4.2 - 2.5 = 1.7 \qquad (C.5)$$

Subgrade Resilient Modulus

Pavement subgrade quality is defined in terms of its resilient modulus M_R. The resilient modulus M_R is a basic material property that can be measured directly in the laboratory, evaluated in-situ from nondestructive tests, or estimated using various empirical relations as detailed in Chapter 5. The 1993 AASHTO Design Guide also incorporates a procedure for considering seasonal fluctuations in M_R to determine a seasonally averaged value for use in design. This procedure is summarized in Section 5.4.3.

Layer Properties

The material properties required for each layer are the structural layer coefficients a_i and, for unbound materials, the drainage coefficients m_i. Methods for evaluating the a_i and m_i values for unbound materials are detailed in Sections 5.4.5 and 5.5.1, respectively. The chart in Figure C-1 can be used to estimate the structural layer coefficient for asphalt concrete in terms of its elastic modulus at 68°F. Values of a_1 between 0.4 and 0.44 are typically used for dense graded asphalt concrete.

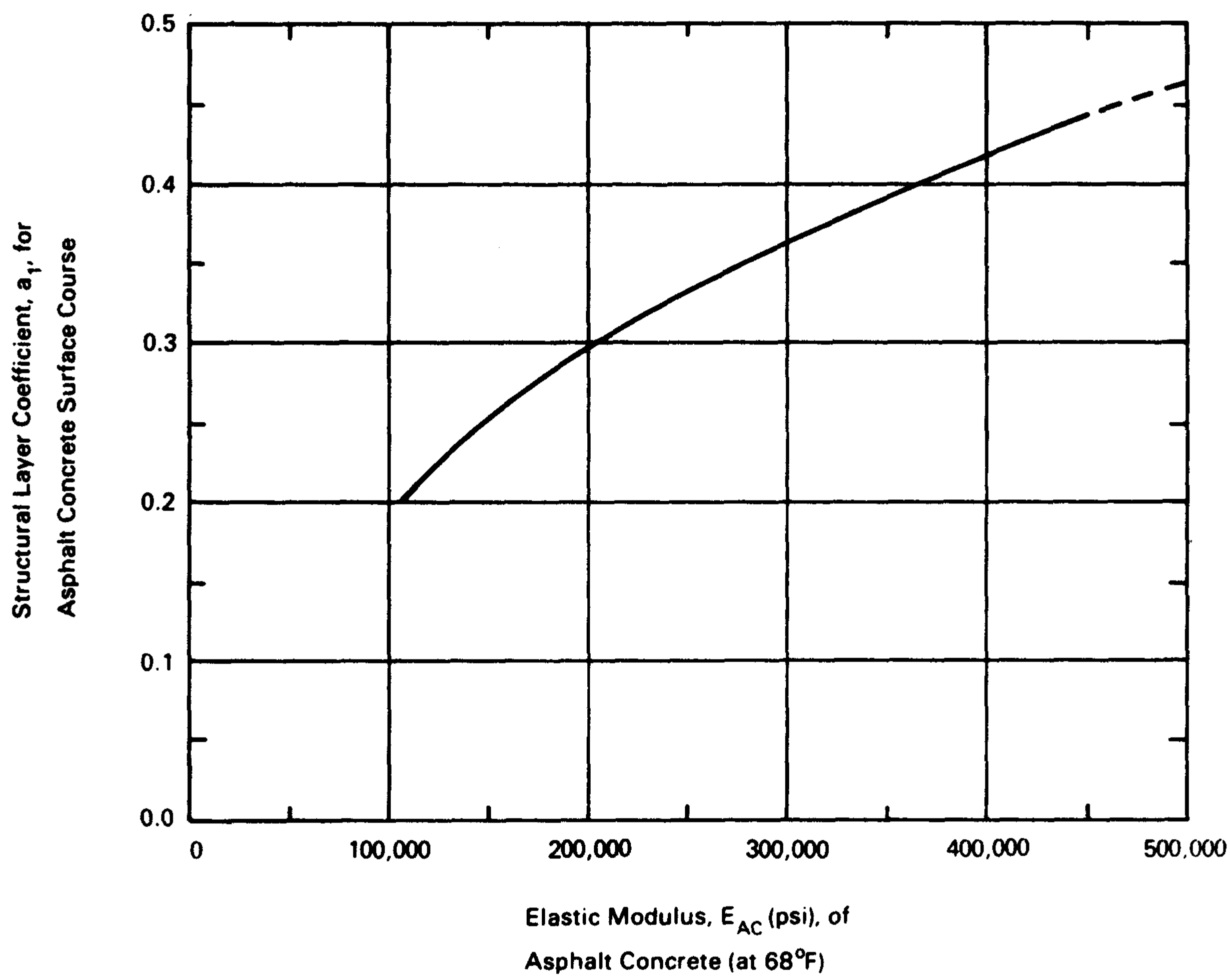


Figure C-1. Chart for estimating structural layer coefficient of dense-graded asphalt concrete based on the elastic (resilient) modulus (AASHTO, 1993).

Procedure

The steps in the 1993 AASHTO flexible pavement design procedure are summarized below in the context of the example baseline scenario presented in Section 6.2.1:

1. Determine the analysis period. For the example design scenario, a 30-year design life is specified.

2. Evaluate the design traffic: W_{18} = 11.6 million ESALs.

3. Determine the design reliability factors: Reliability = 90% (usually set by agency policy), Z_R = -1.282, S_0 = 0.45.

4. Determine the allowable serviceability loss due to traffic: ΔPSI = 1.7 (this may be reduced if frost heave or swelling soils are an issue).

5. Evaluate the seasonally averaged subgrade resilient modulus M_R using the procedures described in Section 5.4.3: M_R = 7,500 psi.

6. Determine the layer properties:
   - Structural layer coefficients a_i for all bound layers (see Section 0 for asphalt concrete, 1993 AASHTO Guide for other stabilized materials) and unbound layers (Section 5.4.5). Recommendations for appropriate a_i values for rehabilitation design are given in Table 5-44 in Section 5.4.5. Values for example design: a_1 = 0.44, a_2 = 0.17.
   - Drainage coefficients m_i for all unbound layers (Section 5.5.1): m_2 = 1.0.

7. Solve Eq. (C.2) for the required overall structural number: SN = 5.07.

8. Determine the design layer thicknesses for the pavement section:
   - Using Eq. (C.2) with M_R set equal to the granular base resilient modulus E_{BS} = 40,000 psi (from the correlation in Eq. 5.16), solve for the required structural number for the asphalt concrete surface layer: SN_1 = 2.62.
   - Convert SN_1 to the required thickness of asphalt: $D_1 = \dfrac{SN_1}{a_1} = 5.95 \rightarrow 6$ inches.[1]

[1]After rounding to the nearest half-inch, per the recommendations in the 1993 AASHTO Design Guide. Unbound layer thicknesses are rounded to the nearest inch.

- Assign the remaining required structural number to the granular base layer: $SN_2 = SN - D_1 a_1 = 2.43$.
- Convert SN_2 to the required thickness of granular base: $D_2 = \frac{SN_2}{m_2 a_2} = 14.3 \rightarrow 14$ inches.[1]

C.3 RIGID PAVEMENT STRUCTURAL DESIGN

Design Equation

The empirical expression relating traffic, pavement structure, and pavement performance for rigid pavements is:

$$\log_{10}\left(W_{18}\right) = Z_R S_o + 7.35\log_{10}\left(D+1\right) - 0.06 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.5-1.5}\right]}{1+\frac{1.64\text{x}10^7}{\left(D+1\right)^{8.46}}} + \left(4.22-0.32p_t\right)\log_{10}\left[\frac{S_c C_d\left(D^{0.75}-1.132\right)}{215.63J\left[D^{0.75}-\frac{18.42}{\left(E_c/k\right)^{0.25}}\right]}\right] \tag{C.6}$$

in which:

W_{18}	=	number of 18 kip equivalent single axle loads (ESALs)
Z_R	=	standard normal deviate (function of the design reliability level)
S_0	=	overall standard deviation (function of overall design uncertainty)
ΔPSI	=	allowable serviceability loss at end of design life
p_t	=	terminal serviceability
k	=	modulus of subgrade reaction (pci)
S_c	=	PCC modulus of rupture (psi)
E_c	=	PCC modulus of elasticity (psi)
J	=	an empirical joint load transfer coefficient
C_d	=	an empirical drainage coefficient
D	=	required PCC slab thickness (inches)

The first ten parameters typically are the inputs to the design equation, and D is the output. Equation (C.6) must be solved implicitly for the slab thickness D as a function of the input parameters.

The design of JRCP and CRCP pavements also requires design of the steel reinforcement. Reinforcement design is beyond the scope of this manual; refer to the 1993 AASHTO Guide for details on this.

Design Inputs

Analysis Period

Same as for flexible pavements; see Section 0.

Traffic

Same as for flexible pavements; see Section 0. Note that the truck factor T_f will not in general be the same for rigid and flexible pavements. Refer to the 1993 AASHTO Design Guide or standard pavement engineering textbooks like Huang (2004) for determination of the truck factor.

Reliability

Similar to flexible pavements; see Section 0. For rigid pavements, values for S_0 typically range between 0.3 and 0.45, with a value of 0.35 commonly used for design.

Serviceability

Similar to flexible pavements; see Section 0. A typical value of p_o for rigid pavements is 4.4. As for flexible pavements, a terminal serviceability index of 2.5 or higher is recommended for design of major highways. Thus, a typical allowable serviceability loss due to traffic for rigid pavements can be expressed as:

$$\Delta PSI = p_t - p_o = 4.4 - 2.5 = 1.9 \tag{C.7}$$

Modulus of Subgrade Reaction

The design modulus of subgrade reaction k is a computed quantity that is a function of the following properties:

- Subgrade resilient modulus M_R
- Thickness of granular subbase D_{SB}
- Resilient modulus of granular subbase E_{SB}
- Depth to bedrock D_{SG} (if shallower than 10 feet)

- Loss of Service *LS* (an index of the erodibility of the granular subbase)

See Section 5.4.6 for the procedure for determining the design value for the modulus of subgrade reaction *k*.

Other Layer Properties

Other layer properties include the modulus of rupture S_c and elastic modulus E_c for the Portland cement concrete slabs, an empirical joint load transfer coefficient *J*, and the subbase drainage coefficient C_d. The PCC parameters S_c and E_c are standard material properties; mean values should be used for the pavement design inputs. The joint load transfer coefficient *J* is a function of the shoulder type and the load transfer condition between the pavement slab and shoulders; recommended values are summarized in Table C-4. See Section 5.5.1 for determination of the drainage coefficient C_d.

Table C-4. Recommended load transfer coefficients for various pavement types and design conditions (AASHTO, 1993).

	No Shoulders		Asphalt Shoulders		Tied PCC Shoulders	
	With Load Transfer Devices	Without Load Transfer Devices	With Load Transfer Devices	Without Load Transfer Devices	With Load Transfer Devices	Without Load Transfer Devices
JPCP/ JRCP	3.2	3.8 – 4.4	3.2	3.8 – 4.4	2.5 – 3.1	3.6 – 4.2
CRCP	2.9	N.A.	2.9 - 3.2	N.A.	2.3 – 2.9	N.A.

Procedure

The steps in the 1993 AASHTO rigid pavement design procedure are summarized below in the context of the example baseline scenario presented in Section 6.2.1:

1. Determine the analysis period. For the example design scenario, a 30-year design life is specified.

2. Evaluate the design traffic: W_{18} = 18.9 million ESALs

.

3. Determine the design reliability factors: Reliability = 90% (usually set by agency policy), Z_R = -1.282, S_0 = 0.45.

4. Determine the terminal serviceability and allowable serviceability loss due to traffic: $p_t = 2.5$, $\Delta PSI = 1.9$ (this may be reduced if frost heave or swelling soils are an issue).

5. Evaluate the effective modulus of subgrade reaction k using the procedures described in Section 5.4.6. Specific design inputs to this procedure are the seasonally averaged subgrade resilient modulus $M_R = 7{,}500$ psi, the assumed thickness of the granular subbase D_{SB}, the seasonally averaged subbase resilient modulus $E_{SB} = 40{,}000$ psi, the depth to bedrock D_{SG} (if less than 10 feet—not the case for this example design), and the loss of service coefficient $LS = 2$.

6. Specify the PCC properties: $S_c = 690$ psi, $E_c = 4.4 \times 10^6$ psi (these would typically be from material specifications; mean values should be used for inputs).

7. Determine the other input parameters: joint load transfer coefficient $J = 3.2$, drainage coefficient $C_d = 1.0$.

8. Solve Eq. (C.6) for the required slab thickness: $D = 10.55 \cong 10.5$ inches.

Note that the thickness assumed for the granular subbase in Step 5 can influence the required slab thickness computed in Step 8. If desired, several design alternatives can be evaluated to arrive at the optimal design.

C.4 SOFTWARE

The empirical design equations for flexible and rigid pavements in Eqs. (C.2) and (C.6) are implicit relationships for the required structural number SN and slab thickness D, respectively. Consequently, an iterative solution algorithm is required. The 1993 AASHTO Design Guide provides nomographs for the graphical evaluation of these equations. They can also be evaluated easily using a spreadsheet, *e.g.,* via the Solver tool in Microsoft Excel. DARWin, a comprehensive software program tied to the 1993 AASHTO Design Guide procedures, is also available through AASHTO. Additional information on DARWin can be found at http://darwin.aashtoware.org/index.htm.

C.5 REFERENCES

AASHTO (1993). *AASHTO Guide for Design of Pavement Structures*, American Association of State Highway and Transportation Officials, Washington, D.C.

Huang, Y.H. (2004). *Pavement Analysis and Design (2^{nd} ed.)*, Prentice-Hall, Englewood Cliffs, NJ.

APPENDIX D: NCHRP 1-37A DESIGN METHOD

D.1 INTRODUCTION

The *Guide for the Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures* developed under NCHRP Project 1-37A is the state-of-the-art procedure for the design of flexible and rigid pavement structures. The mechanistic-empirical approach at the heart of the NCHRP 1-37A methodology represents a fundamental paradigm shift for pavement design. In the mechanistic-empirical approach, the response of the pavement – defined in terms of stresses, strains, and other parameters – is analyzed using rigorous theories of mechanics. Critical response quantities – *e.g.,* tensile strains at the bottom of an asphalt or PCC layer – are then related empirically to pavement performance – *e.g.,* fatigue cracking.

Figure D-1 provides a flow chart for the mechanistic-empirical design approach as implemented in the NCHRP 1-37A procedures. The major steps are

1. Define the traffic, environmental, and other general design inputs for the project. In the case of rehabilitation designs, this will also include information on existing pavement conditions (*e.g.,* distress survey, FWD testing).
2. Select a trial pavement section for analysis. For rehabilitation designs, this includes identification of an appropriate rehabilitation strategy.
3. Define the properties for the materials in the various pavement layers.
4. Analyze the pavement response (temperature, moisture, stress, strain) due to traffic loading and environmental influences. The pavement response analysis is performed on a season-by-season basis in order to include variations in traffic loading, environmental conditions, and material behavior over time.
5. Empirically relate critical pavement responses to damage and distress for the pavement distresses of interest. Damage/distress are determined on a season-by-season basis and then accumulated over the design life of the pavement.
6. Adjust the predicted distresses for the specified design reliability.
7. Compare the predicted distresses at the end of design life against design limits. If necessary, adjust the trial pavement section and repeat Steps 3-7 until all predicted distresses are within design limits.

The corresponding major components required to implement this mechanistic-empirical pavement design methodology are

- Inputs—traffic, climate, materials, others.
- Pavement response models—to compute critical responses.

- Performance models or transfer functions—to predict pavement performance over the design life.
- Design reliability and variability—to add a margin of safety for the design.
- Performance criteria—to set objective distress limits against which the pavement performance will be judged.
- Software—to implement the mechanistic-empirical models and calculations in a usable form.

Each of these components will be briefly summarized in the following sections. Readers should refer to the NCHRP 1-37A final reports (NCHRP 1-37A, 2004) for more thorough coverage of each topic. In addition, Chapter 5 provides detailed information on the geotechnical inputs to the NCHRP 1-37A procedure and Chapter 6 gives several example applications.

D.2 INPUTS

D.2.1 Hierarchical Inputs

As described in Chapter 5, the NCHRP 1-37A design methodology incorporates a hierarchical approach for specifying all pavement design inputs. The hierarchical approach is based on the philosophy that the level of engineering effort exerted in determining design inputs should be commensurate with the relative importance, size, and cost of the design project. Three levels are provided for the design inputs in the NCHRP 1-37A procedure:

Level 1 inputs provide the highest level of accuracy and the lowest level of uncertainty. Level 1 design inputs would typically be used for heavily trafficked pavements or whenever there are serious safety or economic consequences of early failure. Level 1 material inputs require field or laboratory evaluation. Subgrade resilient modulus measured from FWD testing in the field or triaxial testing in the laboratory is one example of a Level 1 input.

Level 2 inputs provide an intermediate level of accuracy and are closest to the typical procedures used with the AASHTO Design Guides. This level could be used when resources or testing equipment are not available for Level 1 characterization. Level 2 inputs would typically be derived from a limited testing program or estimated via correlations or experience (possibly from an agency database). Subgrade resilient modulus estimated from correlations with measured CBR values is one example of a Level 2 input.

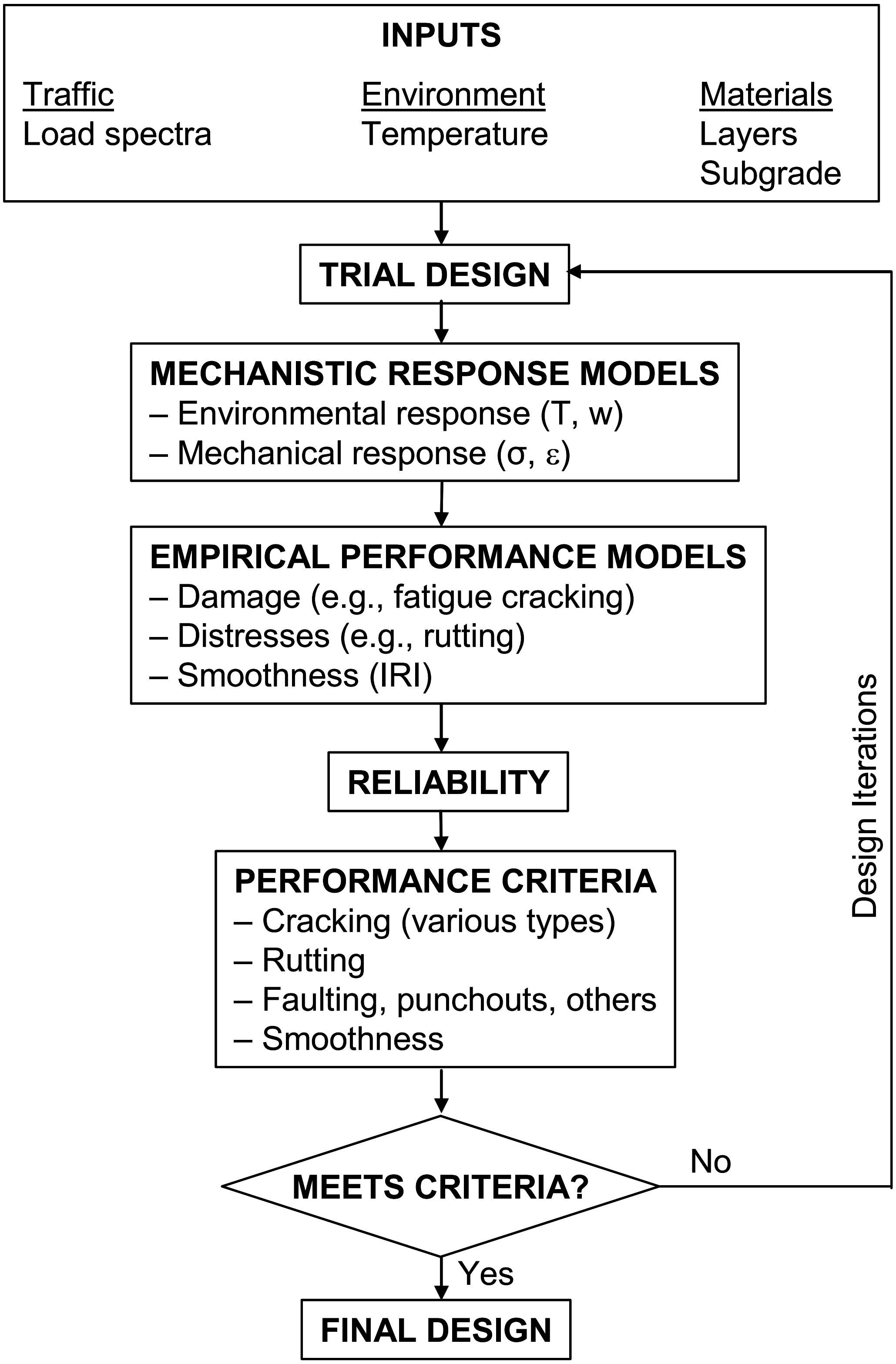


Figure D-1. Flow chart for mechanistic-empirical design methodology.

Level 3 inputs provide the lowest level of accuracy. This level might be used for designs in which there are minimal consequences of early failure (*e.g.,* low volume roads). Level 3 material inputs typically are default values that are based on local agency experience. A default subgrade resilient modulus based on AASHTO soil class is an example of a Level 3 input.

Any given pavement design may incorporate a mix of input data of different levels. For example, measured HMA dynamic modulus values used with default resilient modulus values for the unbound materials in the pavement structure. However, the algorithms used in the design computations are *identical* for all input levels. In other words, the NCHRP 1-37 methodology features levels of input data but not levels of design analysis. The composite input level determines the overall accuracy and reliability of the pavement performance predictions used to judge the acceptability of a trial design.

D.2.2 Traffic

Traffic data are key inputs for the analysis and design of pavement structures. Most existing design procedures, including all of the AASHTO Design Guides, quantify traffic in terms of equivalent single axle loads (ESALs). However, the mechanistic pavement response models in the NCHRP 1-37A methodology require the specification of the magnitudes and frequencies of the actual wheel loads that the pavement is expected to see over its design life. Consequently, traffic must be specified in terms of load spectra rather than ESALs. Load spectra are the frequency distributions of axle load magnitudes by axle configuration (single, tandem, tridem, quad) and season of year (monthly, typically).

State highway agencies typically collect two categories of traffic data. Weigh-in-motion (WIM) data provide information about the number and configuration of axles observed within a set of load groups. Automatic vehicle classification (AVC) data provide information about the number and types of vehicles that use a given roadway as counted over some period of time. **Error! Reference source not found.** summarizes the WIM and AVC data that are required at each of the hierarchical input levels in the NCHRP 1-37A methodology.

The traffic data required in the NCHRP 137A methodology are the same for all pavement types (flexible or rigid) and construction types (new or rehabilitated). Four categories of traffic data are required:

- Traffic volume—base year information
  - Two-way annual average daily truck traffic (AADTT)
  - Number of lanes in the design direction
  - Percent trucks in design direction

    - Percent trucks in design lane
    - Vehicle (truck) operational speed
- Traffic volume adjustment factors
    - Monthly adjustment
    - Vehicle class distribution (see Table 6-5 for an example)
    - Hourly truck distribution (see Table 6-6 for an example)
    - Traffic growth factors
- Axle load distribution factors by season, vehicle class, and axle type (see Table 6-7 for an example)
- General traffic inputs
- Traffic wander data (mean wheel location and standard deviation of lateral wander; lane width)
- Number axles/trucks (see Table 6-8 for an example)
    - Axle configuration (axle width and spacing; tire spacing and pressure)
    - Wheelbase spacing distribution (rigid pavements only; see Table 6-11 for an example)

The NCHRP 1-37A design software takes all of these traffic inputs and computes the number of applications of each axle load magnitude by axle type (single, tandem, tridem, quad) and month. These axle load spectra are a primary input to the mechanistic pavement structural response models.

Table D-1. Traffic data required for each of the three hierarchical input levels (NHCRP, 2004).

Data Sources		Input Level 1	Input Level 2	Input Level 3
Traffic load/volume data	WIM data – site/segment specific	X		
	WIM data – regional default summaries		X	
	WIM data – national default summaries			X
	AVC data – site/segment specific	X		
	AVC data – regional default summaries		X	
	AVC data – national default summaries			X
	Vehicle counts – site/segment specific[1]		X	X
	Traffic forecasting and trip generation models[2]	X	X	X

[1]Level depends on whether regional or national default values are used for the WIM or AVC information.
[2]Level depends on input data and model accuracy/reliability.

D.2.3 Environment

Environmental conditions have a significant effect on the performance of both flexible and rigid pavements. External factors such as precipitation, temperature, freeze-thaw cycles, and depth to water table play key roles in defining the impact of environment on pavement performance. Internal factors such as the susceptibility of the pavement materials to moisture and freeze-thaw damage, drainability of the paving layers, and infiltration potential of the pavement define the extent to which the pavement will react to the external environmental conditions.

Variations in temperature and moisture profiles within the pavement structure and subgrade over the design life of a pavement are simulated in the NCHRP 1-37A design methodology via the Enhanced Integrated Climatic Model (EICM—described more fully in Section D.3.1). The EICM requires a relatively large number of input parameters. As with all other design inputs, EICM input parameters are specified using a hierarchical approach (Levels 1, 2, or 3). Since many of the EICM material property inputs are not commonly measured by most agency and geotechnical laboratories, Level 3 default values will typically be used for most designs. The inputs required by the EICM fall under the following broad categories (see Sections 5.5.2 and 5.6.2 for more detail):

- General information
    - Base/subgrade construction completion date
    - Pavement construction date
    - Traffic opening date
- Weather-related information (Section 5.6.2)
    - Hourly air temperature
    - Hourly precipitation
    - Hourly wind speed
    - Hourly percentage sunshine (used to determine cloud cover)
    - Hourly relative humidity
- Groundwater related information (Section 5.6.2)
    - Groundwater table depth
- Drainage and surface properties (Section 5.5.2)
    - Surface shortwave absorptivity (Section 5.6.2)
    - Infiltration
    - Drainage path length
    - Cross slope
- Pavement materials
    - Asphalt and Portland cement concrete
- Thermal conductivity

- Heat capacity
- Unbound materials (Section 5.5.2)
- Physical properties (specific gravity, maximum dry unit weight, optimum moisture content)
- Soil water characteristic curve
- Hydraulic conductivity (permeability)
- Thermal conductivity
- Heat capacity

The weather-related information required by the EICM can be obtained from weather stations located near the project site. The software accompanying the NCHRP 1-37A Design Guide includes a database from nearly 800 weather stations throughout the United States that can be used to generate the weather-related design inputs.

D.2.4 Material Properties

The material property inputs required for the environmental effects model in the NCHRP 1-37A methodology have already been described in Section D.2.3 (and Sections 5.5.2 and 5.6.2). Additional material property inputs are required for the structural response models used to calculate the stresses and strains in the pavement. As with all other design inputs, the material property inputs can be provided at any of the hierarchical Levels (1, 2, or 3). The material property inputs are most conveniently grouped by material type:

- Asphalt concrete
    - Layer thickness
    - Dynamic modulus (measured value for level 1 or mixture gradation and volumetrics for Level 2 and 3 estimation)
    - Asphalt binder properties (dynamic shear stiffness or viscosity for Levels 1 and 2, binder grade for Level 3)
    - Mixture volumetrics (effective binder content, air voids, unit weight)
    - Poisson's ratio
    - Thermal cracking properties (low temperature tensile strength, creep compliance, thermal expansion coefficient)
- Portland cement concrete
    - Layer thickness
    - Mixture properties (cement and aggregate type, cement content, water/cement ratio, unit weight)
    - Shrinkage characteristics
    - Elastic modulus

- Poisson's ratio
- Compressive strength
- Modulus of rupture
- Thermal expansion coefficient

- Unbound materials (see Sections 5.3 and 5.4 for more details)
  - Material type
  - Layer thickness
  - Unit weight
  - Coefficient of lateral earth pressure
  - Resilient modulus (see Section 5.4.3 for details on inputs at different hierarchical levels)
  - Poisson's ratio

D.2.5 Other

A variety of other input data are required for the NCHRP 1-37A methodology. Some of these inputs are dependent upon the particular pavement type (flexible vs. rigid) and construction type (new vs. rehabilitation) being considered. A brief summary of these other inputs are as follows:

- General project information
  - Design life
  - Latitude, longitude, and elevation (for accessing weather station database)
- Rigid pavement design features (all rigid pavement types)
  - Permanent curl/warp effective temperature difference
  - Base erodibility index
- JPCP design features
  - Joint spacing, sealant type
  - Dowel bar diameter, spacing
  - Edge support (*e.g.,* tied shoulder, widened slab)
  - PCC-base interface bond condition
- CRCP design features
  - Shoulder type
  - Reinforcement (steel percentage, diameter, depth)
  - Mean crack spacing
- Flexible pavement distress potential (new construction)
  - Block cracking
  - Longitudinal cracks outside wheel paths
- Pre-rehabilitation distresses (overlay over AC surface)
  - Rutting

- Fatigue cracking within wheel path
    - Longitudinal cracks outside wheel path
    - Patches
    - Potholes
- Pre-rehabilitation distresses (overlay over PCC surface)
    - Percent cracked slabs before, after restoration
    - CRCP punchouts
    - Dynamic modulus of subgrade reaction

Note that no design features are included for jointed reinforced concrete pavements (JRCP). The NCHRP 1-37A methodology does not include a design capability for this pavement type.

D.3 PAVEMENT RESPONSE MODELS

There are two types of pavement response models in the NCHRP 1-37A methodology: (a) an environmental effects model for simulating the time- and depth-dependent temperature and moisture conditions in the pavement structure in response to climatic conditions; and (b) structural response models for determining the stresses and strains at critical locations in the pavement structure in response to traffic loads. The same environmental effects model is used for all pavement types. Different structural response models are employed for rigid vs. flexible pavements because of the fundamental differences in their mechanical behavior.

D.3.1 Environmental Effects

Diurnal and seasonal fluctuations in the moisture and temperature profiles in the pavement structure induced by changes in groundwater table, precipitation/infiltration, freeze-thaw cycles, and other external factors are incorporated in the NCHRP 1-37A design methodology via the Enhanced Integrated Climatic Model (EICM). The EICM is a mechanistic one-dimensional coupled heat and moisture flow analysis that simulates changes in the behavior and characteristics of pavement and subgrade materials induced by environmental factors. The EICM consists of three major components:

- The Climatic-Materials-Structural Model (CMS Model) originally developed at the University of Illinois (Dempsey *et al*., 1985).
- The CRREL Frost Heave and Thaw Settlement Model (CRREL Model) originally developed at the United States Army Cold Regions Research and Engineering Laboratory (CRREL) (Guymon *et al*., 1986).
- The Infiltration and Drainage Model (ID Model) originally developed at Texas A&M University (Lytton *et al.*, 1990).

Each of these components has been enhanced substantially for use in the NCHRP 1-37A design methodology.

For flexible pavements, the EICM evaluates the following environmental effects:

- Seasonal changes in moisture content for all subgrade and unbound materials.
- Changes in resilient modulus, M_R, of all subgrade and unbound materials caused by changes in soil moisture content.
- Changes M_R due to freezing and subsequent thawing and recovery from frozen conditions.
- Temperature distributions in bound asphalt concrete layers (for determining the temperature-dependent asphalt concrete material properties).

For rigid pavements, the following additional environmental effects are simulated by the EICM:

- Temperature profiles in PCC slabs (for thermal curling prediction).
- Mean monthly relative humidity values (for estimating moisture warping PCC slabs).

One of the important outputs from the EICM for both flexible and rigid pavement design is a set of adjustment factors for unbound layer materials that account for the effects of environmental conditions such as moisture content changes, freezing, thawing, and recovery from thawing. This factor, denoted F_{env}, varies with position within the pavement structure and with time throughout the analysis period. The F_{env} factor modifies the resilient modulus at optimum moisture and density conditions M_{Ropt} to obtain the seasonally adjusted resilient modulus M_R as a function of depth and time.

D.3.2 Structural Response

The mechanistic structural response models determine the stresses, strains, and displacements within the pavement system caused by traffic loads and as influenced by environmental conditions. Environmental influences may be direct (*e.g.,* strains due to thermal expansion and/or contraction) or indirect (*e.g.,* changes in material properties due to temperature and/or moisture effects).

Flexible Pavements

Two flexible pavement analysis methods have been implemented in the NCHRP 1-37A computational procedures. For cases in which all materials in the pavement structure can realistically be treated as linearly elastic, multilayer elastic theory (MLET) is used to determine the pavement response. MLET provides an excellent combination of analysis capabilities, theoretical rigor, and computational speed for linear pavement analyses. In cases where the consideration of unbound material nonlinearity is desired (*i.e.,* Level 1 resilient modulus for new construction), a nonlinear finite element (FE) methodology is employed instead for determining the pavement stresses, strains, and displacements.

A major advantage of MLET solutions is very quick computation times. Solutions for multiple wheel loads can be constructed from the fundamental axisymmetric single wheel solutions via superposition automatically by the computer program. The principal disadvantage of MLET is its restriction to linearly elastic material behavior. Real pavement materials, and unbound materials, in particular, often exhibit stress-dependent stiffness. The materials may even reach a failure condition in some locations, such as in tension at the bottom of the unbound base layer in some pavement structures. These nonlinearities vary both vertically through the thickness of the layer and horizontally within the layer. Some attempts have been made in the past to incorporate these material nonlinearity effects into MLET solutions in an approximate way, but the fundamental axisymmetric formulation of MLET makes it impossible to include the spatial variation of stiffness in a realistic manner.

Some of the limitations of MLET solutions are the strengths of FE analysis. In particular, finite element methods can simulate a wide variety of nonlinear material behavior; the underlying finite element formulation is not constrained to linear elasticity, as is the case with MLET. Stress-dependent stiffness and no-tension conditions for unbound materials can all be treated within the finite element framework. However, the FE computational times are substantially longer than for MLET analyses.

The choice of MLET vs. FE structural response model is made automatically by the NCHRP 1-37A software based on the input data from the user (*i.e.,* whether Level 1 new construction inputs are specified for the unbound resilient modulus values). In both cases, the NCHRP 1-37A software automatically pre-processes all of the input data required for the analysis (*e.g.,* automatically generates a finite element mesh), automatically performs the season-by-season analyses over the specified pavement design life, and automatically post-processes all of the analysis output data to compute the season-by-season values of the critical pavement responses for subsequent use in the empirical performance prediction models.

Performance prediction requires identification of the locations in the pavement structure where the critical pavement responses (stress or strain) attain their most extreme values. For multilayer flexible pavement systems, these locations can be difficult to determine. Critical responses are evaluated at several depth locations in the NCHRP 1-37A analyses, depending upon the distress type:

- Fatigue Depth Locations:
  - Surface of the pavement (z = 0),
  - 0.5 inches from the surface (z = 0.5),
  - Bottom of each bound or stabilized layer.
- Rutting Depth Locations:
  - Mid-depth of each layer/sub-layer,
  - Top of the subgrade,
  - Six inches below the top of the subgrade.

The horizontal locations for the extreme values of critical responses are more difficult to determine. The critical location for the simplest case of a single wheel load can usually be determined by inspection – *e.g.,* directly beneath the center of the wheel. The critical location under multiple wheels and/or axles will be a function of the wheel load configuration and the pavement structure. Mixed traffic conditions (single plus multiple wheel/axle vehicle types) further complicate the problem, as the critical location within the pavement structure will not generally be the same for all vehicle types. The NCHRP 1-37A calculations address this problem by evaluating the pavement responses for a set of potential critical locations. Damage/distress magnitudes are calculated from the pavement responses at each location, with the final performance prediction based on the location having the maximum damage/distress at the end of the analysis period.

Rigid Pavements

Finite element analysis has been proven a reliable tool for computing rigid pavement structural responses. However, the season-by-season distress/damage calculations implemented in the NCHRP 1-37A procedure requires hundreds of thousands of calculations to compute incremental damage over a design period of many years. These computations would take days to complete using existing rigid pavement finite element programs. To reduce computer time to a practical level, neural network models have been developed from a large parametric study performed using the ISLAB2000 finite element program (Khazanovich *et al.*, 2000). The neural network models, which, in effect, are similar to regression models, make it possible to accurately compute critical stresses and deflections virtually instantaneously. This in turn makes it possible to perform detailed month-by-month incremental analysis within a practical timeframe (*i.e.,* a few minutes). Appendix QQ in the NCHRP 1-37A final report (NCHRP, 2004) provides a detailed description of the finite element models, parametric study, and neural networks used for the structural analysis of rigid pavements.

A key feature of the rigid pavement structural response model is its treatment of the pavement foundation. The ISLAB2000 analysis program and the neural network models derived from it employ a modified version of the conventional slab-on-Winkler springs pavement structural model (also called a "dense liquid" foundation model). As shown in Figure D-2, the actual multi-layer pavement structure is replaced by an equivalent 2-layer (slab and base) pavement section resting on a Winkler spring foundation having a stiffness characterized by *k*, the modulus of subgrade reaction (see Section 5.4.6). The effective *k* value in the equivalent 2-layer pavement is determined by matching the computed surface deflections for the actual multi-layer pavement section. The surface deflection profile of the actual section is determined using MLET, modeling all layers in the structure. This computed deflection profile is then used to backcalculate the effective *k* value for the equivalent 2-layer section. Thus, the effective *k* value is an internally computed value, not a direct input to the design procedure. The exception to this is rehabilitation design, where *k* determined from FWD testing may be input directly.

The effective *k* value used in the NCHRP 1-37A methodology is interpreted as a dynamic *k* value (e.g., as determined from FWD testing), which should be distinguished from the traditional static *k* values used in previous AASHTO design procedures.

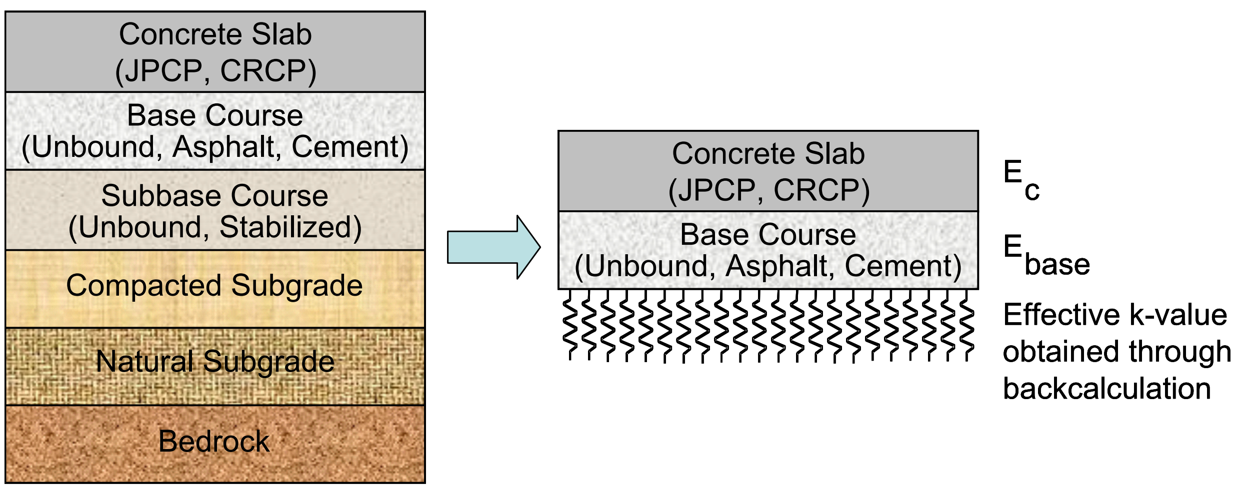


Figure D-2. Structural model for rigid pavement structural response computations.

D.4 PAVEMENT PERFORMANCE MODELS

Pavement performance is evaluated in terms of individual distress modes in the NCHRP 1-37A methodology. A variety of empirical distress models – also sometimes termed "transfer functions" – are incorporated in the NCHRP 1-37A methodology for the major structural distresses in flexible and rigid pavements. Empirical models are also provided for estimating smoothness as a function of the individual structural distresses and other factors.

D.4.1 Damage vs. Distress

Some distresses can be evaluated directly during the season-by-season calculations. For example, the empirical model for rutting in the asphalt layers in flexible pavements is of the form:

$$\frac{\varepsilon_p}{\varepsilon_r} = \beta_{r1} a_1 T^{a_2 \beta_{r2}} N^{a_3 \beta_{r3}} \tag{D.1}$$

in which:

- ε_p = accumulated plastic strain after N repetitions of load at the critical location
- ε_r = resilient strain at the critical location
- N = number of load repetitions
- T = temperature
- a_i = regression coefficients derived from laboratory repeated load permanent deformation tests
- β_{ri} = field calibration coefficients (see Section D.4.4)

Each asphalt layer is divided into sublayers, and Eq. (D.1) is evaluated at the midthickness of each sublayer. The contribution ΔR_{d_i} to total rutting R_d from sublayer i having thickness h_i can then be expressed as:

$$\Delta R_{d_i} = \varepsilon_{p_i} . \Delta h_i \tag{D.2}$$

The contributions of all of the sublayers l can then be summed to give the total rutting for the asphalt concrete layer:

$$R_d = \sum_{i=1}^{l} \Delta R_{d_i} \tag{D.3}$$

Other distresses cannot be evaluated directly, but must be quantified in terms of computed damage factors. For example, the empirical model for "alligator" fatigue cracking in the asphalt layers in flexible pavements is of the form:

$$N_f = \beta_{f1} k_1 (\varepsilon_t)^{-\beta_{f2} k_2} (E)^{-\beta_{f3} k_3} \tag{D.4}$$

in which:

N_f	=	number of repetitions to fatigue cracking failure
ε_t	=	tensile strain at the critical location
E	=	asphalt concrete stiffness (at appropriate temperature)
k_1, k_2, k_3	=	regression coefficients determined from laboratory fatigue tests
β_{f1}, β_{f2}, β_{f3}	=	field calibration coefficients (see Section D.4.4)

Computation of fatigue damage is based upon Miner's Law:

$$D = \sum_{i=1}^{T} \frac{n_i}{N_{fi}} \tag{D.5}$$

in which:

D	=	damage
T	=	total number of seasonal periods
n_i	=	actual traffic for period i
N_{fi}	=	traffic repetitions causing fatigue failure under conditions prevailing during period i

The damage factor determined using Eq. (D.5) is then related to observed fatigue distress quantities (*e.g.,* area of fatigue cracking within the lane) during the field calibration process (Section D.4.4).

D.4.2 Distress Models

Empirical distress prediction models are provided for the following structural distresses in the NCHRP 1-37A flexible pavement design methodology:

- Permanent deformation (rutting)
  - o Within asphalt concrete layers
  - o Within unbound base and subbase layers
  - o Within the subgrade
- Fatigue cracking
  - o Within asphalt concrete layers
- Bottom-up (classical "alligator" cracking)
- Top-down (longitudinal fatigue cracking)
  - o Within cement stabilized layers
- Thermal cracking

The empirical structural distress models for rigid pavements include

- Transverse joint faulting (JPCP)
- Transverse fatigue cracking (JPCP)
- Punchouts (CRCP)

Note that reflection cracking for asphalt concrete overlays is not included in the current version of the NCRHP 1-37A methodology. At the time of the NCHRP 1-37A development, it was judged that no suitable empirical reflection cracking models yet existed. It is anticipated that a suitable model will be developed and added to the NCHRP 1-37A procedure in the future.

D.4.3 Smoothness

Pavement smoothness is often used as a composite index of pavement quality. Smoothness (or loss thereof) is influenced by nearly all of the distresses of interest in flexible and rigid pavement systems. Smoothness data is also regularly and routinely collected and stored as part of the pavement management systems at many agencies. Lastly, smoothness is directly related to overall ride quality, the factor of most importance to highway users. Because of these reasons, empirical smoothness prediction models have been incorporated in the NCHRP 1-37A design methodology.

Pavement smoothness in the NCHRP 1-37A models is characterized in terms of the International Roughness Index, or IRI. IRI is predicted as a function of the initial as-constructed IRI, the subsequent development of distresses over time, and other factors such as subgrade type and climatic conditions that may affect smoothness through mechanisms such as shrinkage or swelling of subgrade soils and frost heave. The structural distresses influencing smoothness are predicted directly by the NCHRP 1-37A mechanistic-empirical methodology. However, nonstructural distresses cannot be evaluated using mechanistic-empirical principles, so the NCHRP 1-37A procedure provides the option of specifying the overall potential for these other distresses. Smoothness loss due to soil shrinking/swelling/frost heave and other climatic factors are incorporated into the NCHRP 1-37A IRI models through the use of a "site factor."

The NCHRP 1-37A design method provides IRI prediction models as a function of pavement type (flexible vs. rigid), base type (flexible pavements), and construction type (new vs. rehabilitation). IRI models are provided for the following cases:

- AC (new construction)
    - AC over granular base
    - AC over asphalt-treated base
    - AC over cement-stabilized material
- AC overlay (rehabilitation)
    - AC over flexible pavement
    - AC over rigid pavement
- JPCP (new construction)
- JPCP (rehabilitation)
    - JPCP restoration
    - Bonded PCC over JPCP
    - Unbonded PCC over JPCP
- CRCP (new construction)
- CRCP (rehabilitation)
    - CRCP restoration
    - Bonded PCC over JPCP
    - Unbonded PCC over CRCP (rehabilitation)

Appendix OO in the NCHRP 1-37A final documentation (NCHRP, 2004) provides a detailed description of the development of these models.

D.4.4 Field Calibration

The distress prediction models are key components of the NCHRP 1-37A mechanistic-empirical design and analysis procedure. Calibration of these models against field performance is an essential part of the model development. Calibration refers to the mathematical process by which the models are adjusted to minimize the differences between predicted and observed values of distress.

All performance models in the NCHRP 1-37A design method have been calibrated on a global level to observed field performance at a representative set of pavement test sites around North America. Test sections from the FHWA Long Term Pavement Performance (LTPP) program were used extensively in the calibration process because of the consistency of the monitored data over time and the diversity of test sections throughout North America.

However, there were some serious limitations to the NCHRP 1-37A field calibration. Many of the material property and site feature inputs required for the NCHRP 1-37A analyses were unavailable from the LTPP database. Because of the limited number of pavement test sites with complete input data, the minimal material testing available, the use of calculated properties from correlations (*i.e.,* Level 3 inputs), and the global scope of the calibration effort, the predictions from the calibrated models still have relatively high levels of uncertainty and a limited inference space of application. The recently completed NCHRP Project 9-30 (Von Quintus *et al*., 2003) has formulated a plan for developing an enhanced database for future recalibration of the NCHRP 1-37A and other similar pavement models.

The NCHRP 1-37A software also includes a provision for entering local or regional field calibration factors instead of the national values derived from the LTPP database. This feature permits local agencies to adjust the mechanistic-empirical performance predictions to better reflect their local conditions.

D.5 DESIGN RELIABILITY

A large amount of uncertainty and variability exists in pavement design and construction, as well as in the traffic loads and climatic factors acting over the design life. In the NCHRP 1-37A mechanistic-empirical design, the key outputs of interest are the individual distress quantities. Therefore, variability of the predicted distresses is the focus of design reliability.

The incorporation of reliability in the NCHRP 1-37A procedure is similar in some respects to the way it is treated in the 1993 AASHTO Guide. In the 1993 AASHTO Guide, an overall standard deviation or "uncertainty" is specified for the design inputs (the S_0 value—see Appendix C), a desired reliability level is selected based on agency policy, and the combination of the standard deviation and reliability are then used in essence to add a "margin of safety" to the design traffic W_{18}. The NCHRP 1-37A methodology differs from the 1993 AASHTO procedure in that the standard deviations and reliability levels are set for each individual distress mode predicted in the mechanistic-empirical computations. The default value for the standard deviation of each predicted distress quantity is based on a careful analysis of the differences between the predicted versus actual distresses during the field calibration of the empirical performance models (Section D.4.4). These estimates of error represent the combined effects of input variability, variability in the construction process, and model error.

The desired level of reliability is specified along with the acceptable level of distress at the end of design life (Section D.6) to define the performance requirements for a pavement design in the NCHRP 1-37A procedure. For example, one criterion might be to limit the percent of cracked PCC slabs to 8% at a design reliability of 90%. Then, on average for 100 projects, 90 would be expected to exhibit fewer than 8% slabs cracked at the end of the design life. Different reliability levels may be specified for different distresses in the same design. For example, the designer may choose to specify 95% reliability for slab cracking, but 90% reliability for faulting and IRI. Of course, increasing design reliability will lead to more substantial pavement sections and higher initial costs. The beneficial trade-off is that future maintenance costs should be lower for the higher-reliability design.

D.6 PERFORMANCE CRITERIA

Performance criteria are definitions of the maximum amounts of individual distress or smoothness acceptable to an agency at a given reliability level. Performance criteria are a user input in the NCHRP 1-37A methodology and depend on local design and rehabilitation policies. Default performance criteria built into the current version of the NCHRP 1-37A software are summarized in Table D-2. The designer can select all or some subset of the performance criteria to be evaluated during the design.

D.7 SOFTWARE

The mechanistic-empirical calculations in the NCHRP 1-37A design methodology cannot be performed by hand or simple spreadsheets. A Windows-based program has been developed to implement the NCHRP 1-37A methodology by providing: (1) an interface to input all design variables, (2) computational engines for analysis and performance prediction, and (3) results and outputs from the analyses in formats suitable for use in electronic documents or for making hard copies.

Table D-2. Default performance criteria in NCHRP 1-37A software.

Distress	Unit	Limit[1]
Flexible Pavements		
Top-down (longitudinal) fatigue cracking	feet/mile	1000
Bottom-up (alligator) fatigue cracking	% of wheel path area	25
Thermal fracture	feet/mile	1000
Chemically stabilized layer fatigue cracking	% of wheel path area	25
Total permanent deformation (rutting)	inch	0.75
Permanent deformation (rutting) in asphalt layer	inch	0.25
Terminal IRI[2]	inches/mile	172
Rigid Pavements		
Transverse fatigue cracking (JPCP)	% slabs cracked	15
Mean joint faulting (JPCP)	inch	0.12
Punchouts (CRCP)	number per mile	10
Terminal IRI[2]	inches/mile	172

[1]Default value from software version 0.700 (4/7/2004).

[2]Default initial IRI = 63 inches/mile.

The software presents a series of information and input screens coordinated through a *main program layout* screen, as illustrated in Figure D-3. On this screen, all access points to the information and data input screens are color-coded to guide the designer in providing all data needed to run a design analysis. Green tags indicate screens on which the designer has already entered/reviewed data, yellow tags indicate screens containing default data that have not yet been reviewed/approved by the designer, and red tags indicate screens that have missing required data that must still be entered by the designer before the calculations can be performed. Clicking on any tag brings up the corresponding data input screen; for example, Figure D-4 shows an example data entry screen for subgrade material properties.

The main program layout screen provides access to the following five groupings of information and input screens (*screens* are denoted by the symbol "•", subordinate screen *tabs* by the symbol "♦"):

1. *Project Information*
    - General Information
    - Site/Project Identification
    - Analysis Parameters

2. *Traffic Inputs*

- Traffic Volume Adjustment Factors
  - Monthly Adjustment
  - Vehicle Class Distribution
  - Hourly Distribution
  - Traffic Growth Factors
- Axle Load Distribution Factors
- General Traffic Inputs
  - Number of Axles/Truck
  - Axle Configuration
  - Wheelbase

3. *Climate Inputs*

- Climate

4. *Structure Inputs*

- Structure
  - Drainage and Surface Properties
  - Layers
    - Layer Material Properties
  - Thermal Cracking

5. *Distress Potential*

Note that the *Structure Inputs* listing above is for the case of a new flexible pavement design. The screens will be slightly different for other pavement and construction types, but they all conform to the general organization listed above.

Once all necessary information and input data have been entered into the program, the user clicks the *Run Analysis* button to carry out all the required computations. Separate areas of the main program layout screen provide (1) the status (% complete) of the analyses in progress and (2) links to summary screens for the inputs to the analyses and their results in both tabular and graphical formats. For example, the design analysis of a conventional flexible pavement design might provide output plots of HMA modulus, alligator cracking, thermal cracking, rutting, and IRI versus pavement age. Figure D-5 is an example of the type of output generated by the software. Output can be generated as either Microsoft Excel spreadsheets or as HTML documents for easy import into other engineering applications.

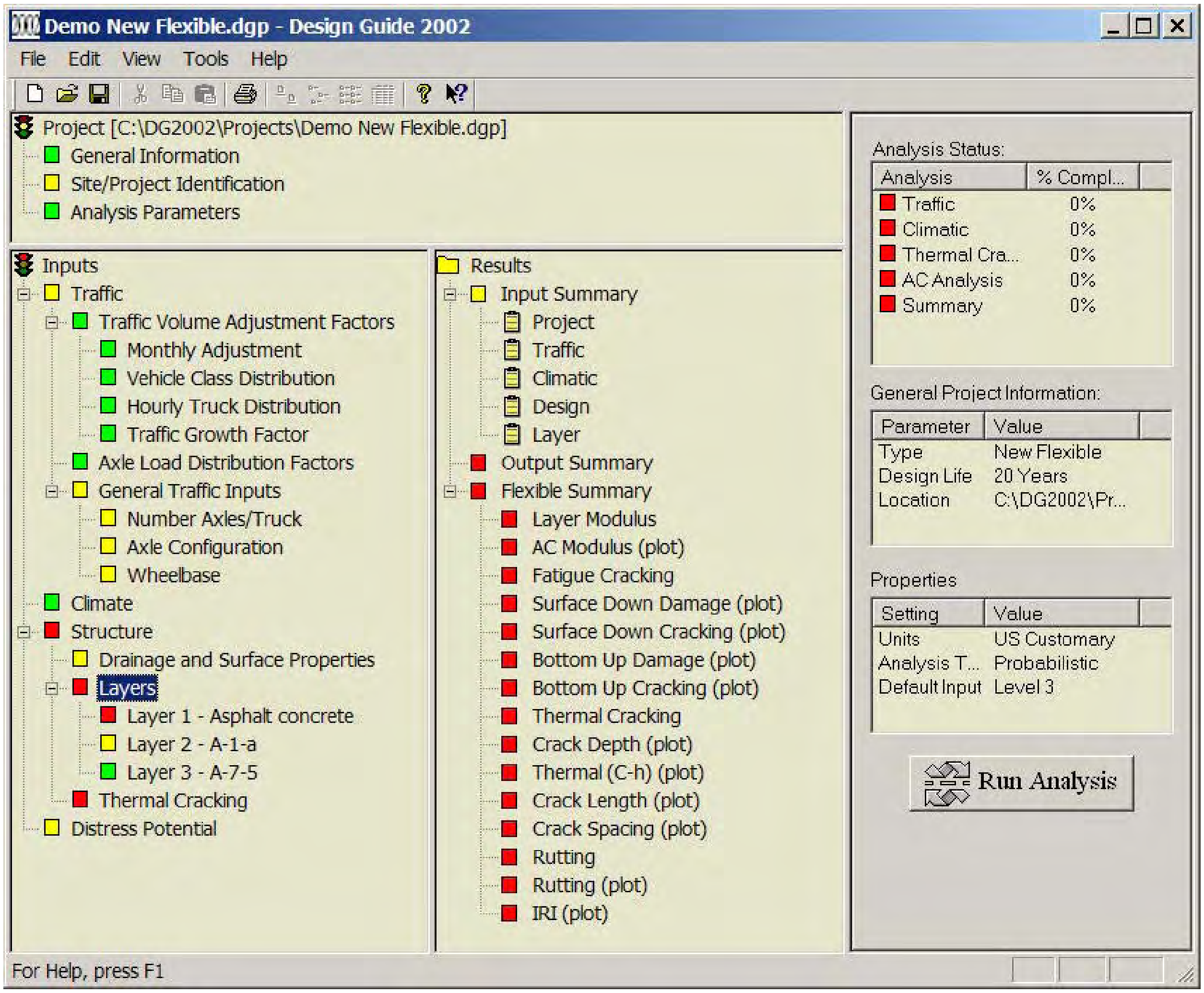


Figure D-3. Main input screen for NCHRP 1-37A software.

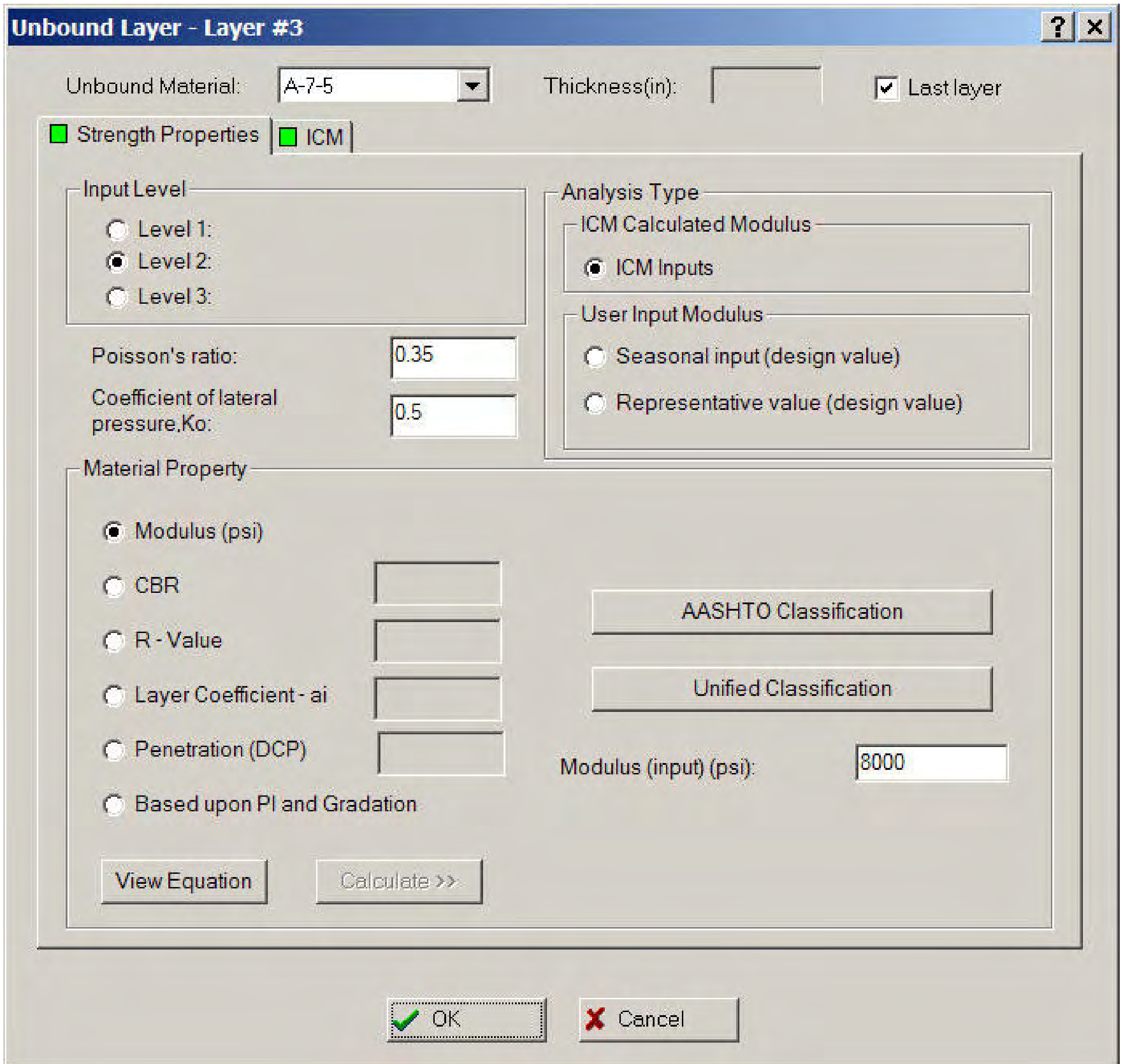


Figure D-4. Typical data entry screen for NCHRP 1-37A software.

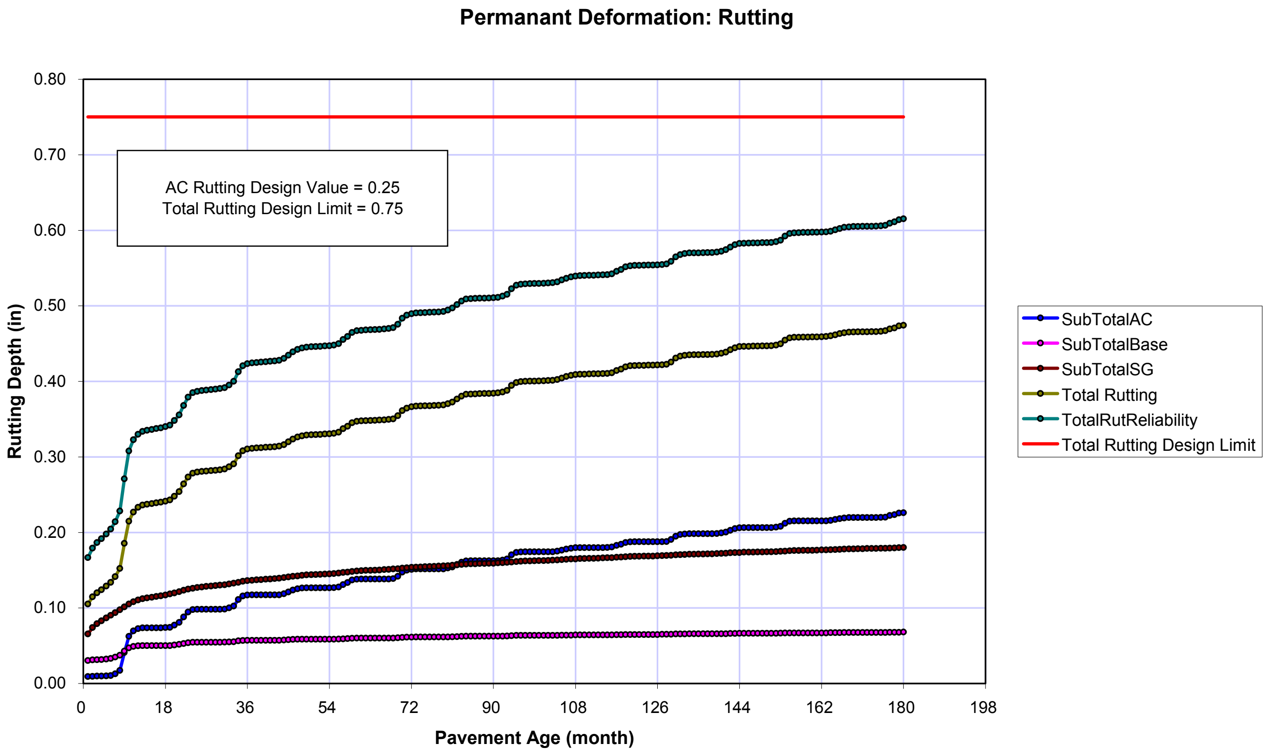


Figure D-5. Typical graphical output from NCHRP 1-37A software.

D.8 REFERENCES

Dempsey, B.J., Herlach, W.A. and Patel, A.J. (1985). *The Climatic-Material-Structural Pavement Analysis Program*, FHWA/RD-84/115, Vol. 3, Final Report, Federal Highway Administration, Washington D.C.

Guymon, G.L., Berg, R.L. and Johnson, T.C. (1986). *Mathematical Model of Frost Heave and Thaw Settlement in Pavement,* Report: U.S. Army Cold Region Research and Engineering Laboratory, Hanover, NH.

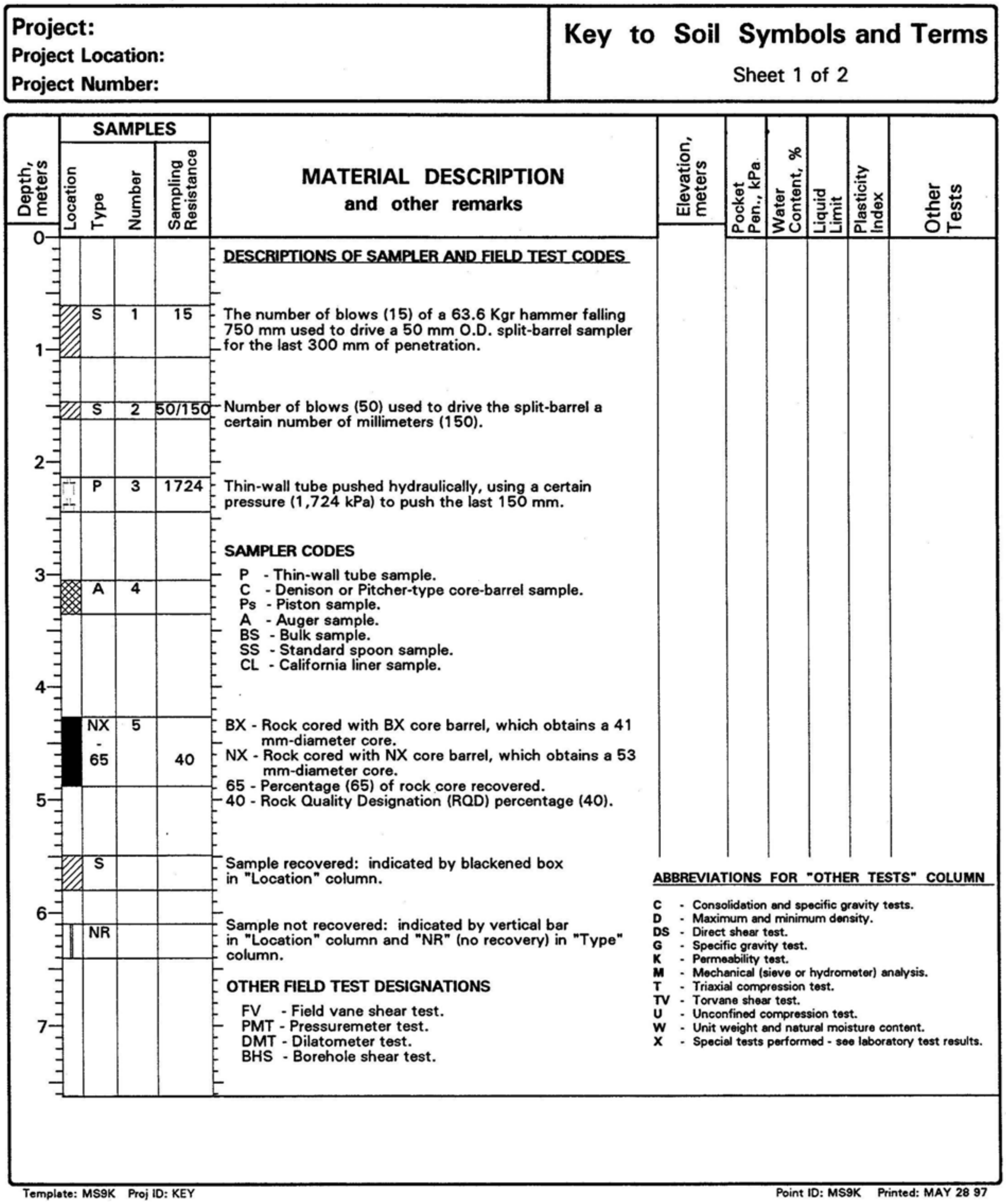
Khazanovich, L., Yu, H.T., Rao, S., Galasova, K., Shats, E., and Jones, R. (2000). *ISLAB2000 - Finite Element Analysis Program for Rigid and Composite Pavements*. User's Guide. ERES Consultants Division of Applied Research Associates, Inc., Champaign, IL.

Lytton, R.L., Pufahl, D.E., Michalak, C.H., Liang, H.S. and Dempsey, B.J. (1990). *An Integrated Model of the Climatic Effects on Pavement,* Texas Transportation Institute, Texas A&M University, Report No. FHWA-RD-90-033, Federal Highway Administration, McLean, VA.

NCHRP 1-37A (2004). *Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures*, Draft Report, Transportation Research Board, National Research Council, Washington, D.C.

Von Quintus, H.L., Schwartz, C.W., McCuen, R.H. and Andrei D. (2003). "Experimental Plan for Calibration and Validation of Hot Mix Asphalt Performance Models for Mix and Structural Design," *Final Report, NCHRP Project 9-30*, Transportation Research Board, National Research Council, Washington, D.C.

APPENDIX E:
TYPICAL KEY FOR BORING LOG PREPARATION



Project:
Project Location:
Project Number:

Key to Soil Symbols and Terms
Sheet 1 of 2

Depth, meters	SAMPLES Location	Type	Number	Sampling Resistance	MATERIAL DESCRIPTION and other remarks	Elevation, meters	Pocket Pen., kPa	Water Content, %	Liquid Limit	Plasticity Index	Other Tests
0					DESCRIPTIONS OF SAMPLER AND FIELD TEST CODES						
1		S	1	15	The number of blows (15) of a 63.6 Kgr hammer falling 750 mm used to drive a 50 mm O.D. split-barrel sampler for the last 300 mm of penetration.						
		S	2	50/150	Number of blows (50) used to drive the split-barrel a certain number of millimeters (150).						
2		P	3	1724	Thin-wall tube pushed hydraulically, using a certain pressure (1,724 kPa) to push the last 150 mm.						
3		A	4		SAMPLER CODES P - Thin-wall tube sample. C - Denison or Pitcher-type core-barrel sample. Ps - Piston sample. A - Auger sample. BS - Bulk sample. SS - Standard spoon sample. CL - California liner sample.						
4		NX - 65	5	40	BX - Rock cored with BX core barrel, which obtains a 41 mm-diameter core. NX - Rock cored with NX core barrel, which obtains a 53 mm-diameter core. 65 - Percentage (65) of rock core recovered. 40 - Rock Quality Designation (RQD) percentage (40).						
5		S			Sample recovered: indicated by blackened box in "Location" column.						
6		NR			Sample not recovered: indicated by vertical bar in "Location" column and "NR" (no recovery) in "Type" column.						
7					OTHER FIELD TEST DESIGNATIONS FV - Field vane shear test. PMT - Pressuremeter test. DMT - Dilatometer test. BHS - Borehole shear test.						

ABBREVIATIONS FOR "OTHER TESTS" COLUMN

C - Consolidation and specific gravity tests.
D - Maximum and minimum density.
DS - Direct shear test.
G - Specific gravity test.
K - Permeability test.
M - Mechanical (sieve or hydrometer) analysis.
T - Triaxial compression test.
TV - Torvane shear test.
U - Unconfined compression test.
W - Unit weight and natural moisture content.
X - Special tests performed - see laboratory test results.

Template: MS9K Proj ID: KEY Point ID: MS9K Printed: MAY 28 97

Figure E-1. Example key to boring log.

Project:	# Key to Soil Symbols and Terms
Project Location:	
Project Number:	Sheet 2 of 2

TERMS DESCRIBING CONSISTENCY OR CONDITION

COARSE-GRAINED SOILS (major portion retained on No. 200 sieve): includes (1) clean gravels and sands and (2) silty or clayey gravels and sands. Condition is rated according to relative density as determined by laboratory tests or standard penetration resistance tests.

Descriptive Term	Relative Density	SPT Blow Count
Very loose	0 to 15%	< 4
Loose	15 to 35%	4 to 10
Medium dense	35 to 65%	10 to 30
Dense	65 to 85%	30 to 50
Very dense	85 to 100%	> 50

FINE-GRAINED SOILS (major portion passing on No. 200 sieve): includes (1) inorganic and organic silts and clays, (2) gravelly, sandy, or silty clays, and (3) clayey silts. Consistency is rated according to shearing strength, as indicated by penetrometer readings, SPT blow count, or unconfined compression tests.

Descriptive Term	Unconfined Compressive Strength, kPa	SPT Blow Count
Very soft	< 25	< 2
Soft	25 to 50	2 to 4
Medium stiff	50 to 100	4 to 8
Stiff	100 to 200	8 to 15
Very stiff	200 to 400	15 to 30
Hard	> 400	> 30

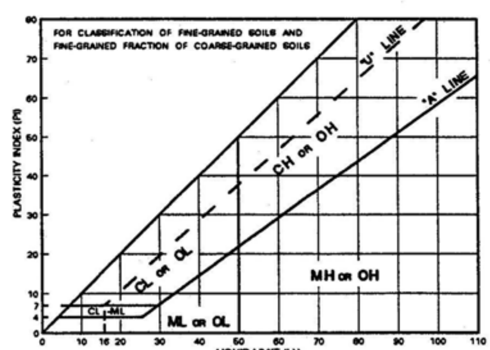
GENERAL NOTES

1. Classifications are based on the Unified Soil Classification System and include consistency, moisture, and color. Field descriptions have been modified to reflect results of laboratory tests where deemed appropriate.

2. Surface elevations are based on topographic maps and estimated locations.

3. Descriptions on these boring logs apply only at the specific boring locations and at the time the borings were made. They are not warranted to be representative of subsurface conditions at other locations or times.

Major Divisions			Group Symbols	Typical Names	Laboratory Classification Criteria	
Coarse-Grained Soils (More than half of material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (Little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve) coarse-grained soils are classified as follows: Less than 5 percent . . . . . . GW, GP, SW, SP; More than 12 percent . . . . . GM, GC, SM, SC; 5 to 12 percent . . . . Borderline cases requiring dual symbols **	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3
			GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for GW
		Gravels with fines (Appreciable amount of fines)	GM* d / u	Silty gravels, gravel-sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4 — Above "A" line with P.I. between 4 and 7 are border-line cases requiring use of dual symbols
			GC	Clayey gravels, gravel-sand-silt mixtures		Atterberg limits above "A" line or P.I. greater than 7
	Sands (More than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (Little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines		$C_u = \frac{D_{60}}{D_{10}}$ greater than 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3
			SP	Poorly-graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for SW
		Sands with fines (Appreciable amount of fines)	SM* d / u	Silty sands, sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4 — Above "A" line with P.I. between 4 and 7 are border-line cases requiring use of dual symbols
			SC	Clayey sands, sand-clay mixtures		Atterberg limits above "A" line or P.I. greater than 7
Fine-Grained Soils (More than half of material is smaller than No. 200 sieve size)	Silts and Clays (Liquid limit less than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity	Plasticity Chart	
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays		
			OL	Organic silts and organic silty clays of low plasticity		
	Silts and Clays (Liquid limit greater than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts		
			CH	Inorganic clays of high plasticity, fat clays		
			OH	Organic clays of medium to high plasticity, organic silts		
	Highly Organic Soils		Pt	Peat and other highly organic soils		



Plasticity Chart

Particle Size

Material		Sieve size	mm
Silt or clay		< #200	< 0.074
Sand	Fine	#200 to #40	0.074 to 0.42
	Medium	#40 to #10	0.42 to 2.00
	Coarse	#10 to #4	2.00 to 4.76
Gravel	Fine	#4 to 3/4 in.	4.76 to 19.1
	Coarse	3/4 in. to 3 in.	19.1 to 76.2
Cobbles		3 in. to 12 in.	76.2 to 304.8
Boulders		12 in. to 36 in.	304.8 to 914.4

* Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is based on Atterberg Limits: suffix d used when L.L. is 28 or less and the P.I. is 6 or less; the suffix u used when L.L. is greater than 28.

** Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group symbols. For example: GW-GC, well-graded gravel-sand mixture with clay binder.

Figure E-1. Example key for final boring log (continued).

Project: Project Location: Project Number:	**Key to Rock Core Log** Sheet 1 of 2

Depth, meters	Elevation, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Frac. Freq.	R Q D, %	Fracture Drawing/ Number		Lithology	MATERIAL DESCRIPTION	Packer Tests	Laboratory Tests	Drill Rate, meters/hour	FIELD NOTES
0														
1	2	3	4	5	6	7	8	9	10	11 META-ARKOSE, light gray, moderately weathered, moderately strong.	13	14	15	16
2		1	1	100	1	80				12 a b c d e f g h				Slow drilling
					0			1 M		1: 75, J, VN, Fe, Su, Pl, S, VC M: Mechanical Breakage				
4														

1	**Depth:**	Distance (in meters) from the collar of the borehole.
2	**Elevation:**	Elevation (in meters) from the collar of the borehole.
3	**Run No.:**	Number of the individual coring interval, starting at the top of bedrock.
4	**Box No.:**	Number of the core box which contains core from the corresponding run.
5	**Recovery:**	Amount (in percent) of core recovered from the coring interval; calculated as the length of core recovered divided by the length of the run.
6	**Frac. Freq.:**	(Fracture Frequency) The number of naturally occurring fractures in each foot of core; does not include mechanical breaks, which are considered to be induced by drilling.
7	**R Q D:**	(Rock Quality Designation) Amount (in percent) of intact core (pieces of sound core greater than 100 mm in length) in each coring interval; calculated as the sum of the lengths of intact core divided by the length of the core run.
8	**Fracture Drawing:**	Sketch of the naturally occurring fractures and mechanical breaks, showing the angle of the fractures relative to the cross-sectional axis of the core. "NR" indicates no recovery.
9	**Fracture Number:**	Location of each naturally occurring fracture (numbered) and mechanical break (labeled "M"). Naturally occurring fractures are described in Column 11 (keyed by number) using descriptive terms defined on the following page (Items a - h).
10	**Lithology:**	A graphic log presentation using symbols to represent differing rock types.
11	**Description:**	Lithologic description in this order: rock type, color, texture, grain size, foliation, weathering, strength, and other features; descriptive terms are defined on the following page. A detailed descriptive log of overburden materials is not necessarily provided.
12	**Discontinuity Description:**	Abbreviated description of fracture corresponding to number of naturally occurring fracture in Column 9 using terms defined on the following page (Items a - h).
13	**Packer Tests:**	A vertical line depicts the interval over which a packer test is performed.
14	**Laboratory Tests:**	A vertical line depicts the interval over which core has been removed for laboratory testing. Laboratory tests performed are indicated in Column 16.
15	**Drill Rate:**	Rate (in meters per hour) of penetration of drilling. "N/O" indicates rate not observed.
16	**Field Notes:**	Comments on drilling, including water loss, reasons for core loss, and use of drilling mud; also, laboratory tests performed on core.

Template: MASK Proj ID: KEY Print ID: COREKEY Printed: MAY 28 97

Figure E-2. Example key to core boring log.

Project:	Key to Rock Core Log
Project Location:	Sheet 2 of 2
Project Number:	

Depth, meters	Elevation, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Frac. Freq.	R Q D, %	Fracture Drawing/ Number	Lithology	MATERIAL DESCRIPTION	Packer Tests	Laboratory Tests	Drill Rate, meters/hour	FIELD NOTES

KEY TO DESCRIPTIVE TERMS USED ON CORE LOGS

DISCONTINUITY DESCRIPTORS

a Dip of fracture surface measured relative to horizontal

b **Discontinuity Type:**
- F - Fault
- J - Joint
- Sh - Shear
- Fo - Foliation
- V - Vein
- B - Bedding

c **Discontinuity Width (millimeters):**
- W - Wide (12.5-50)
- MW - Moderately Wide (2.5-12.5)
- N - Narrow (1.25-2.5)
- VN - Very Narrow (<1.25)
- T - Tight (0)

d **Type of Infilling:**
- Cl - Clay
- Ca - Calcite
- Ch - Chlorite
- Fe - Iron Oxide
- Gy - Gypsum/Talc
- H - Healed
- No - None
- Py - Pyrite
- Qz - Quartz
- Sd - Sand

e **Amount of Infilling:**
- Su - Surface Stain
- Sp - Spotty
- Pa - Partially Filled
- Fi - Filled
- No - None

f **Surface Shape of Joint:**
- Wa - Wavy
- Pl - Planar
- St - Stepped
- Ir - Irregular

g **Roughness of Surface:**
- Slk - Slickensided [surface has smooth, glassy finish with visual evidence of striations]
- S - Smooth [surface appears smooth and feels so to the touch]
- SR - Slightly Rough [asperities on the discontinuity surfaces are distinguishable and can be felt]
- R - Rough [some ridges and side-angle steps are evident; asperities are clearly visible, and discontinuity surface feels very abrasive]
- VR - Very Rough [near-vertical steps and ridges occur on the discontinuity surface]

h **Discontinuity Spacing (meters):**
- EW - Extremely Wide (>20)
- W - Wide (7-20)
- M - Moderate (2.5-7)
- C - Close (0.7-2.5)
- VC - Very Close (<0.7)

ROCK WEATHERING / ALTERATION

Description	Recognition
Residual Soil	Original minerals of rock have been entirely decomposed to secondary minerals, and original rock fabric is not apparent; material can be easily broken by hand
Completely Weathered/Altered	Original minerals of rock have been almost entirely decomposed to secondary minerals, minerals, although original fabric may be intact; material can be granulated by hand
Highly Weathered/Altered	More than half of the rock is decomposed; rock is weakened so that a minimum 50-mm-diameter sample can be broken readily by hand across rock fabric
Moderately Weathered/Altered	Rock is discolored and noticeably weakened, but less than half is decomposed; a minimum 50-mm-diameter sample cannot be broken readily by hand across rock fabric
Slightly Weathered/Altered	Rock is slightly discolored, but not noticeably lower in strength than fresh rock
Fresh	Rock shows no discoloration, loss of strength, or other effect of weathering/alteration

ROCK STRENGTH

Description	Recognition	Approximate Uniaxial Compressive Strength (kPa)
Extremely Weak Rock	Can be indented by thumbnail	250 - 1,000
Very Weak Rock	Can be peeled by pocket knife	1,000 - 5,000
Weak Rock	Can be peeled with difficulty by pocket knife	5,000 - 25,000
Medium Strong Rock	Can be indented 5 mm with sharp end of pick	25,000 - 50,000
Strong Rock	Requires one hammer blow to fracture	50,000 - 100,000
Very Strong Rock	Requires many hammer blows to fracture	100,000 - 250,000
Extremely Strong Rock	Can only be chipped with hammer blows	> 250,000

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Figure E-2. Example key to core boring log (continued).

APPENDIX F:

DETERMINATION OF ADMIXTURE CONTENT FOR SUBGRADE STABILIZATION

(Adopted from Joint Departments of the Army and Air Force, USA,
TM 5-822-14/AFMAN 32-8010, *Soil Stabilization for Pavements*,
25 October 1994.)

Lime Content for Lime-Stabilized Soils

To determine the design lime content for a subgrade soil, the following steps are suggested:

1. Determine whether the soil has at least 25% passing the 75-µm sieve and has a plasticity index (PI) of at least 10. The soil screening criteria also limit soluble sulfates to less than 0.3 % by weight in a 10:1 water-to-soil solution.
2. Determine the initial design lime content by mixing varying amounts of lime with the soil in water and measuring the pH levels in 1-hour intervals. Select the lowest lime mixture level for which a pH of 12.4 occurs as the initial design lime content.
3. Using the initial design lime content conduct moisture-density tests to determine the maximum dry density and optimum water content of the soil lime mixture defined by the user agency, *e.g.,* AASHTO T-99, AASHTO T-180, ASTM D 698, or ASTM D 1557. The procedures in ASTM D 3551 will be used to prepare the soil-lime mixture.
4. Prepare specimens at optimum moisture content and specified density requirement (*e.g.,* 90% of AASHTO T-180) using the initial design lime content and at about 2% and 4% lime above that lime content from Step 1. Cure the test specimens in sealed plastic bags for 28 days at 21^{o}C (73^{o}F). (Alternative – cure for 7 days at 40^{o} C (104^{o}F)).
5. Determine the unconfined compressive strength for all cured test specimens (*e.g.,* ASTM 5102). Select as the construction design lime content the minimum percent required to achieve the required compressive strength (*e.g.,* 150 psi). Either prepare a sample at the design lime content and perform resilient modulus test (*e.g.,* AASHTO T 294-94) or estimate from Unconfined compression strength Q_u. A conservative estimate for lime-stabilized soils has been reported to be obtained from (Thompson, 1970):

$$M_R = 0.124\, q_u + 9.98$$

where,

M_R = resilient modulus, ksi,

q_u = unconfined compressive strength, psi, as tested in accordance with

ASTM D 5102, "Standard Test Method for Unconfined Compressive Strength of Compacted Soil-Lime Mixtures"

6. Add 0.5 – 1% additional lime in the lower percentage ranges to compensate for problems associated with non-uniform mixing during construction.

Laboratory testing should always be performed to check whether the stabilization has the desired effect on other engineering properties like plasticity and strength.

Cement Content for Cement-Modified Soils

(1) *Improve plasticity.* The amount of cement required to improve the quality of the soil through modification is determined by the trial-and-error approach. If it is desired to reduce the PI of the soil, successive samples of soil-cement mixtures must be prepared at different treatment levels and the PI of each mixture determined. The Referee Test of ASTM D 423 and ASTM D 424 procedures will be used to determine the PI of the soil-cement mixture. The minimum cement content that yields the desired PI is selected, but since it was determined based upon the minus 40 fraction of the material, this value must be adjusted to find the design cement content based upon total sample weight expressed as:

$$A = 100BC$$

where,

A = design cement content, percent total weight of soil
B = percent passing No. 40 sieve size, expressed as a decimal
C = percent cement required to obtain the desired PI of minus 40 material, expressed as a decimal

(2) *Improve gradation.* If the objective of modification is to improve the gradation of a granular soil through the addition of fines, then particle-size analysis (ASTM D 422) should be conducted on samples at various treatment levels to determine the minimum acceptable cement content.

(3) *Reduce swell potential.* Small amounts of Portland cements may reduce swell potential of some swelling soils. However, Portland cement generally is not as effective as lime, and may be considered too expensive for this application. The determination of cement content to reduce the swell potential of fine-grained plastic soils can be accomplished by molding several samples at various cement contents and soaking the specimens along with untreated specimens for 4 days. The lowest cement content that eliminates the swell

potential or reduces the swell characteristics to the minimum is the design cement content. Procedures for measuring swell characteristics of soils are found in ASTM D 4546 and MIL-STD-621A, Method 101. The cement content determined to accomplish soil modification should be checked to see whether it provides an unconfined compressive strength great enough to qualify for a reduced thickness design in accordance with criteria established for soil stabilization.

(4) *Condition frost areas.* Cement-modified soil may also be used in frost areas, but in addition to the procedures for mixture design described in (1) and (2) above, cured specimens should be subjected to the 12 freeze-thaw cycles prescribed by ASTM D 560 (but omitting wire-brushing) or other applicable freeze-thaw procedures. This should be followed by determination of frost design soil classification by means of standard laboratory freezing tests. If cement-modified soil is used as subgrade, its frost susceptibility, determined after freeze-thaw cycling, should be used as the basis of the pavement thickness design if the reduced subgrade design method is applied.

Cement Content for Cement-Stabilized Soil

The following procedure is recommended for determining the design cement content for cement-stabilized soils.

Step 1. Determine the classification and gradation of the untreated soil following procedures in ASTM D 422 and D 2487, respectively.

Step 2. Using the soil classification, select an estimated cement content for moisture-density tests from Table F-1.

Table F-1. Cement requirements for various soil types.

Soil Type	Initial Estimated Cement Content percent dry weight
GW, SW	5
GP, GW-GC, GW-GM, SW-SC, SW-SM	6
GC, GM, GP-GC, GP-GM, GM-GC, SC, SM, SP-SC, SP-SM, SM-SC, SP	7
CL. ML, MH	9
CH	11

Step 3. Using the estimated cement content, conduct moisture-density tests to determine the maximum dry density and optimum water content of the soil-cement mixture. The procedure contained in ASTM D 558 will be used to prepare the soil-cement mixture and to make the necessary calculations; however, the procedures outlined in AASHTO T180 or ASTM D 1557 will be used to conduct the moisture density test.

Step 4. Prepare triplicate samples of the soil-cement mixture for unconfined compression and durability tests at the cement content selected in Step 2 and at cement contents 2% above and 2% below that determined in Step 2. The samples should be prepared at the density and water content to be expected in field construction. For example, if the design density is 95% of the laboratory maximum density, the samples should also be prepared at 95%. The samples should be prepared in accordance with ASTM D 1632, except that when more than 35% of the material is retained on the 4.75 mm (# 4) sieve, a 100-mm (4-in.) diameter by 200-mm-high (8-in.) mold should be used to prepare the specimens. Cure the specimens for 7 days in a humid room before testing. Test three specimens using the unconfined compression test in accordance with ASTM D 1633, and subject three specimens to durability tests, either wet-dry (ASTM D 559) or freeze-thaw (ASTM D 560) tests, as appropriate. The frost susceptibility of the treated material should also be determined, as indicated in appropriate pavement design manuals.

Step 5. Compare the results of the unconfined compressive strength and durability tests with the requirements. The lowest cement content that meets the required unconfined compressive strength requirement and demonstrates the required durability is the design cement content. If the mixture should meet the durability requirements, but not the strength requirements, the mixture is considered to be a modified soil. If the results of the specimens tested do not meet both the strength and durability requirements, then a higher cement content may be selected and Steps 1 through 4 above repeated.

Selection of Lime-Flyash Content for LF and the Determination of the Ratio of Lime to Fly LCF Mixtures.

(1) *Step 1.* The first step is to determine the optimum fines content that will give the maximum density. This is done by conducting a series of moisture-density tests using different percentages of flyash and determining the mix level that yields maximum density. The initial flyash content should be about 10%, based on dry weight of the mix. It is recommended that material larger than 19 mm (¾ in.) be removed and the test conducted on the minus 19 mm (¾ in.) fraction. Tests are run at increasing increments of flyash, *e.g.,* 2%,

up to a total of about 20%. Moisture density tests should be conducted following procedures indicated in AASHTO T99, AASHT T180, and ASTM D 1557. The design flyash content is then selected at 2% above that yielding maximum density. An alternate method is to estimate optimum water content and conduct single point compaction tests at flyash contents of 10 – 20%, make a plot of dry density versus flyash content, and determine the flyash content that yields maximum density. The design flyash content is 2% above this value. A moisture density test is then conducted to determine the optimum water content and maximum dry density.

(2) *Step 2.* Determine the ratio of lime to flyash that will yield highest strength and durability. Using the design flyash content and the optimum water content determined in Step 1, prepare triplicate specimens at three different lime-flyash ratios, following the selected density procedure. Use LF ratios of 1:3, 1:4, and 1:5. If desired, about 1% of Portland cement may be added at this time.

(3) *Step 3.* Test three specimens using the unconfined compression test. If frost design is a consideration, subject three specimens to 12 cycles of freeze-thaw durability tests (ASTM D 560), except wire brushing is omitted. The frost susceptibility of the treated material shall also be determined as indicated in the appropriate design manual.

(4) *Step 4.* Compare the results of the unconfined compressive strength and durability tests with the requirements. The lowest LF ratio content, *i.e.,* ratio with the lowest lime content that meets the required unconfined compressive strength requirement and demonstrates the required durability, is the design LF content. The treated material must also meet frost susceptibility requirements, as indicated in the appropriate pavement design manuals. If the mixture should meet the durability requirements, but not the strength requirements, it is considered to be a modified soil. If the results of the specimens tested do not meet both the strength and durability requirements, a different LF content may be selected, or additional Portland cement used and Steps 2 through 4 repeated.

Selection of Cement Content for LCF Mixtures.

Portland cement may also be used in combination with LF for improved strength and durability. If it is desired to incorporate cement into the mixture, the same procedures indicated for LF design should be followed except that, beginning at Step 2, the cement shall be included. Generally, about 1 – 2% cement is used. Cement may be used in place of or in addition to lime; however, the total tines content should be maintained. Strength and durability tests must be conducted on samples at various LCF ratios to determine the combination that gives best results.

Selection of Asphalt Content for Bituminous-Stabilized Soil

Guidance for the design of bituminous-stabilized base and subbase courses is contained in U.S. Army TM 5-822-8/AFM 88-6, Chap. 9. For subgrade stabilization, the following equation may be used for estimating the preliminary quantity of cutback asphalt to be selected:

$$p = \frac{0.02(a) + 0.07(b) + 0.15(c)\ 0.20(d)}{(100 - s)}\ x\ 100$$

where

p = percent cutback asphalt by weight of dry aggregate

a = percent of mineral aggregate retained on No. 50 sieve

b = percent of mineral aggregate passing No. 50 sieve and retained on No. 100 sieve

c = percent of mineral aggregate passing No. 100 sieve and retained on No. 200 sieve

d = percent of mineral aggregate passing No. 200 sieve

s = percent solvent

The preliminary quantity of emulsified asphalt to be used in stabilizing subgrades can be determined from Table F-2. The final design content of cutback or emulsified asphalt should be selected based upon the results of the Marshal Stability test procedure (AASHTO T 245, ASTM D 5581, MIL-STD 620A). The minimum Marshall Stability recommended for subgrades is 2.2 kN (500 lb). If a soil does not show increased stability when reasonable amounts of bituminous materials are added, the gradation of the soil should be modified, or another type of bituminous material should be used. Poorly graded materials may be improved by the addition of suitable tines containing considerable material passing the 75 µm (No. 200) sieve. The amount of bitumen required for a given soil increases with an increase in percentage of the liner sizes.

Table F-2. Emulsified asphalt requirements.

Percent Passing 75-µm (No. 200) Sieve	Pounds of Emulsified Asphalt per 100 pounds of Dry Aggregate at Percent Passing No. 10 Sieve					
	<50	60	70	80	90	100
0	6.0	6.3	6.5	6.7	7.0	7.2
2	6.3	6.5	6.7	7.0	7.2	7.5
4	6.5	6.7	7.0	7.2	7.5	7.7
6	6.7	7.0	7.2	7.5	7.7	7.9
8	7.0	7.2	7.5	7.7	7.9	8.2
10	7.2	7.5	7.7	7.9	8.2	8.4
12	7.5	7.7	7.9	8.2	8.4	8.6
14	7.2	7.5	7.7	7.9	8.2	8.4
16	7.0	7.2	7.5	7.7	7.9	8.2
18	6.7	7.0	7.2	7.5	7.7	7.9
20	6.5	6.7	7.0	7.2	7.5	7.7
22	6.3	6.5	6.7	7.0	7.2	7.5
24	6.0	6.3	6.5	6.7	7.0	7.2
25	6.2	6.4	6.6	6.9	7.1	7.3

1 lb = 0.454 kg

Table F-3. Common guidelines for stabilized drainable base mixes
(after FHWA Demonstration Project 87: Drainable Pavement Systems, FHWA-SA-92-008).

Stabilization Method	Item	Requirement
Asphalt-Stabilized	Gradation of material	AASHTO No. 67 stone, preheat at 135^o – 160^o C (275^o – 320^o F).
	Amount of asphalt	2 – 2.5% by weight, using a harder asphalt like AC 40 or AR 8000.
	Temperature of mix	Lay at 90^o – 120^o C (195^o – 250^o F) and seal with one pass of a 7.2 – 10.9 metric ton (8 – 12 ton) smooth wheel roller. Start compaction rolling after the temperature reaches 65^o C (150^o F), but before it drops to 38^o C (100^o F).
Cement-Stabilized	Gradation of material	AASHTO No. 67 stone.
	Amount of cement	Use 110 – 150 kg of cement per cubic meter (185 – 250 lbs/yd^3). (135 – 150 kg/m^3 (230 – 250 lbs/yd^3) for high traffic loads). (A minimum compressive strength of 4.1 MPa (600 psi) is typically suggested in cold regions to resist frost deterioration.)
	Curing requirements	Not clearly understood, and may require local testing (consider a 150 m-long (500 ft) test strip). It is suggested that the mix be covered with plastic for five days after laydown, or that light misting be done, starting the second day after laydown.